IIb. Means for increasing the tensile strength and for reducing cracking of concrete

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Means for Increasing the Tensile Strength and for Reducing Cracking of Concrete.

Mittel zur Erhöhung der Zugfestigkeit
und zur Verminderung der Rissebildung des Betons.

Moyens d'augmenter la résistance à la traction et de diminuer
la formation des fissures dans le béton.

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Encreasing the Tensile Strength and Avoidance Formation of Cracks in Concrete.

Erhöhung der Zugfestigkeit und Verminderung der Rißbildung des Betons.

La résistance à la traction et la fissuration du béton.

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Introduction.

It has long been the aim of material-research workers to find means of avoiding cracks in concrete and of keeping unavoidable cracks within safe limits. This aim has become even more important with the development in the practical application of concrete and reinforced concrete and the efforts made to attain higher stress limits. In this connection emphasis must always be laid on the close relation between the formation of cracks and the tensile strength of the concrete, and on the necessity of increasing the latter. The object of the following report is to review this field of inquiry in the light of the knowledge and experience at present available.

I. Tensile strength of concrete.

a) Measuring the tensile strength of concrete.

The tensile strength of concrete can be measured either directly by means of tensile tests, or indirectly by means of bending tests. Tensile tests are but seldom employed, since they can only be carried out with the help of expensive testing apparatus and test pieces which are difficult to make, besides which they require much more care in execution than do bending tests. In addition to which comes the fact that bending tests mostly give a better reproduction of the stressing to which the concrete is actually subjected than do tensile tests.

In both forms of test the result is dependent upon the cross section of the pieces, i. e. on the size of the latter, since as a rule the larger the piece the smaller the strengths obtained (1 p. 84). The reason for this is primarily to be sought in the self-stresses which are set up as, for instance, the test piece dries out (1 p. 87) (cf. I c 8). In bending tests, moreover, the arrangement of the load has to be observed. Two single loads situated at a certain distance apart give on an average a smaller rupture-point bending stress than one single load, because in the case of two single loads the greatest amount of stressing is distributed over the whole section between them and is thus more likely to hit on the weakest places in the concrete (1 p. 93). The rupture-point stresses calculated from tensile and bending tests carried out on the same concrete and with the customary assumptions (uniformly distributed stress for tensile tests, stress of linear proportion for bending tests) do not coincide; the rupture-point bending stress is even greater than the tensile strength. The principal reason for this is that in the tensile test the greatest stress between the points of attack of the loads is set up simultaneously at all points of a cross section, whereas in the bending test it appears at first only in the extreme fibres (1 p. 93). Moreover, there exists no proportionality between stresses and strains in the tensile zone of the concrete even under slight loading, so that the distribution of stresses does not correspond to the assumptions made in the calculation of the stresses (4 p. 39) (39 p. 73).

b) Relation between tensile, bending and compressive strengths.

Up to the present no law has been discovered to govern the relation between the tensile, bending and compressive strengths of concrete and to allow reliable conclusions to be drawn between one strength and the other. The reports available as to relative values differ largely from each other.

Graf (1 p. 92) found that between the compressive strength, calculated on 30 cm cubes, and the tensile strength of bodies with a cross section of 400 cm², $K_d : K_z$ = from 8 to 17. *Guttmann* (3) found $K_d : K_z = 14$ to 28 for cubes and tensile pieces of 100 cm² cross section. In both cases the difference between the relative values is therefore the same amount, while its absolute amount is obviously influenced by the dimensions of the test pieces.

Graf states that for the relation of the compressive to the bending strength (2 p. 83) $K_d : K_b$ = from 4 to 12; he bases his figures on a large number of tests.

The variations in the relation between bending and tensile strength are correspondingly large. *Graf* (2 p. 91) found that with 400 cm² cross section under tension $K_b : K_z = 1.6$ to 2.9, the maximum value being 3.5 for centrifugally cast concrete, while *Guttmann* (3) gives $K_b : K_z = 2.3$ to 4.2 for 100 cm² cross section and bend stressing caused by one single load. *Dutron* (5) observed that $K_b : K_z = 1.3$ to 2.0 with the same cross section under tension but with bend stressing from two single loads. The influence of the loading arrangement becomes clear from the last two sets of figures, as referred to under Ia.

When considering the variations, however, it must be remembered that mixtures were compared which differ in several factors at the same time. If the number of variables were to be limited, there would probably be more chance of finding some regulated relationship. Hummel's observations on the relations between bending and compressive strengths give him the equation $K_b = K_d^x$ (6 p. 15). From other tests it can be concluded with the same degree of probability that there is a corresponding relation $K_z = K_d^y$ between tensile and compressive strengths. These two equations inform us that tensile and bending strengths increase as compressive strength increases, though not in the same proportion as the latter, but the more slowly the greater the compressive strengths become. As was to be expected from the above-mentioned variations of the ratios $\frac{K_d}{K_b}$ and $\frac{K_d}{K_z}$, x and y are not constant values holding good for all cases. It is even probable that x varies between 0.55 and 0.70 and y between 0.45 and 0.60 (see Table I). Nevertheless, the exponents x

Table 1.

Influence of the composition of concrete on its tensile and bending strength,

Group	Tests by	Aggregates	Fuller sandcurve as per Fig.1	Grading of sand 0-0,2 %	Sand to total aggregates in %	Cement per m³ of concrete kg/m³	Water-cement ratio W	Bending (tensile) strength K_b K_z kg/cm²	Compressive strength K_d kg/cm²	x (y) for $K_b = K_d^x$ $K_z = K_d^y$	Consistency
1	2	3	4	5	6	7	8	9	10	11	12
1	Bach	Natural sand+gravel	below B	2	58	~240	0,82	(12)	138	(0,513)	
	and										very
	Graf		"	2	55	~320	0,61	(17)	201	(0,534)	soft
	(13)										
	p.42		"	2	57	~430	0,50	(23)	264	(0,560)	
2	Graf	Natural sand+gravel	A	—	43	264	0,63	41	278	0,659	soft
			B	22	60	257	0,77	36	183	0,687	
	(11)		below C	40	71	254	0,98	18	133	0,620	Spread
	p.40		C	44	80	250	1,17	13	81	0,586	
											~ 51 cm
			A	—	43	308	0,52	50	301	0,686	
			B	17	59	297	0,65	44	242	0,689	
			below C	37	71	294	0,81	29	170	0,656	
			C	44	79	296	0,96	16	119	0,581	
			A	—	40	353	0,51	48	362	0,657	
			B	12	58	343	0,58	46	256	0,690	
			below C	34	70	351	0,67	37	251	0,654	
			C	44	79	367	0,68	24	166	0,621	
3	Hertel	Natural sand+gravel									soft
	(15)		below B	9	38	306	0,70	29	240	0,616	
			" B	7	46	302	0,70	37	250	0,653	
			above C	15	95	300	1,00	14	90	0,584	
		Natural sand+chips	C	11	37	310	0,72	31	195	0,686	
		(Squat basalt chips)	B	13	41	307	0,72	30	238	0,619	
			above A	8	50	303	0,72	36	244	0,651	
4											
	Guttman	Natural sand+gravel	below B	8	43	278	0,60	(18)	354	(0,484)	soft
	(3)		"	8	54	275	0,60	(21)	353	(0,520)	
		Crushed sand+chips	below B	8	43	301	0,68	(16)	303	(0,486)	
		(basalt)	"	8	54	300	0,68	(21)	305	(0,533)	
5											
	Bach	Natural sand+gravel	below B	2	55	~320	0,61	(17)	201	(0,534)	very soft
	and Graf	Natural sand+chips (basalt)	B	6	56	~350	0,77	(21)	197	(0,573)	
	(13) 28	Crushed sand+gravel (basalt)	below B	13	55	~320	0,90	(17)	157	(0,558)	
		Crushed sand+chips (basalt)	B	17	56	~350	1,05	(16)	124	(0,572)	
6	Graf	Natural sand									
		Natural sand+gravel	above C	5	40	301	0,69	40	265	0,661	less soft
	(17) p.5+6	Natural sand+chips Squat basalt chips	"	6	40	300	0,71	38	193	0,691	Spread
		Natural sand+chips Basalt chips flat	"	5	40	299	0,71	41	227	0,685	~ 40 cm
		Natural sand+gravel	"	5	40	297	0,76	32	207	0,650	liquid
		Natural sand+chips Squat basalt chips	"	6	40	308	0,93	24	109	0,678	Spread
		Natural sand+chips Basalt chips flat	"	5	40	295	0,87	25	155	0,638	~ 67 cm
7											
	Dutrun	Natural sand+gravel				346	0,50	(20)	384	(0,507)	soft
	(5)	Natural sand+chips (Porphyrus)				350	0,55	(21)	335	(0,526)	
		Natural sand+chips Furnace slag				355	0,60	(25)	350	(0,553)	
		Crushed sand+chips (Porphyrus)				365	0,67	(24)	273	(0,569)	
		Crushed sand+chips Furnace slag				360	0,74	(22)	247	(0,561)	
8											
	Walz	Natural sand+gravel	A	2	42	254	0,54	57,5	330	0,698	(1) moist
						250	0,64	51,0	280	0,698	(2)
	(18)										
			A	2	42	345	0,46	63,5	445	0,681	(1)
						343	0,50	59,5	395	0,684	(2)
											1) vibrated 2) rammed

and y enable a more reliable judgment to be formed as to the action of certain measures on the relation of bending or tensile strength to compressive strength than do the simple ratio values between these strengths, because the exponents obviously cope very satisfactorily with the variation in the ratio values, which is dependent upon the compressive strength figure.

c) Influences on the tensile strength of concrete.

If it becomes a question of raising the tensile strength of concrete all factors and possibilities must be investigated that might have an influence on the properties of the material, such as: — the cement, nature of stone, shape and mesh of aggregates, cement-aggregate-water mixture, manner of working up, external conditions during setting and afterwards — e. g. temperature and humidity — and age and loading.

1) Cement.

As the strength of concrete is produced by the binding properties of the cement, these properties are responsible in the first instance for the tensile strength of concrete. This consideration would seem to be contradictory to the fact that the classification of different grades of cement according to the standard tests for tensile strength in general use until a short time ago, is quite a different thing from that yielded by tensile tests on concrete made of these cements: In other words, the fact that cement showing greater tensile strength in the standard tests does not always produce the stronger concrete, as for instance in (3). Conclusions have been drawn from these, and from similar observations made during compression tests to the effect that the traditional standard system of testing with uniformly granulated sand and low water-cement ratio does not represent an adequate standard of evaluation for the binding properties of cements made up into concrete; new methods of testing, with sand of mixed granule sizes and a higher water-cement ratio, were therefore developed. (7) to (10) Experiments made by the Forschungsgesellschaft für das deutsche Straßenwesen (German Institute for Road Research) have revealed fair coincidence between the relations of the bending strengths of various cements as given by the new testing method, and the bending strengths of concrete made with these cements. This supplies proof that the bending strength of concrete can be increased by the employment of a cement that the new testing methods have proved to be superior. Furthermore, it has now become possible to investigate the reasons why certain cements are superior to others. Judging by the knowledge which we possess today as regards the action of cements, however, it is hardly to be expected that the quality of cement in respect of the tensile strength of concrete can be increased to any great extent above present-day standards.

The relation between the bending and the compressive strength of concrete also varies, according to the new testing method, between widely-differing limits and as a rule works out the more unfavourably as regards bending strength, the greater the compressive strength (10). The relation between these two strengths in concrete is thus influenced by the specific properties of the cement also.

To what extent the consistency of the paste is important in determining the amount of water necessary for the attaining of a certain degree of workability of the concrete, and how far the tendency to shrink and the time taken in setting affect its tensile strength, will be discussed later on in another connection (cf. S. 3 and 8).

2) Quantity of water in concrete.

For the same aggregates and consistency of the concrete, its tensile or bending strength increases with the amount of cement it contains (11 p. 48) (12). This increase can be followed in Table 1, group 1, and also in group 2, by comparing the values belonging to the same Fuller curve. It can be concluded from the alteration of the exponents y in group 1 that the relation $\frac{K_z}{K_d}$ becomes more favourable as the cement content increases. It probably approaches the specific value for the respective brand of cement. In group 2, on the other hand, x is partly constant. The reason for this would seem to lie in the higher content, graduated according to the amount of cement, in the granulation 0 to 0.2 mm, whereby the cement is not merely used as a filler, even in the case of lean mixtures. But as the quantity of cement increases, so do the internal stresses produced by the drying out process (cf. S. 8), since the sections dry more slowly (25 p. 34) so that the strengths may drop for a time (12) in spite of a higher cement content.

3) Quantity of water in concrete.

The quantity of water in fresh concrete influences the tensile and bending strength in just the same manner as it does the compressive strength. As the water-cement ratio

$$w = \frac{\text{weight of water}}{\text{weight of cement}}, \text{ increases,}$$

the tensile and bending strength drops, being dependent upon w; *Graf* puts this relation at approximately $\frac{1}{w^2}$ (2 p. 86). Owing to the relation mentioned under 2b) between the strengths, the loss of strength as w increases is of course comparatively smaller for tensile and bending strength than for compressive strength. To reduce the quantity of water necessary for the attaining of a certain workability of concrete, the use of liquid aggregates (14) may be found advantageous — in addition, of course, to the choice of a suitable brand of cement (cf. S. 1) and proper granulation (cf. S. 4).

4) Granulation of the aggregates.

As the granulation of the aggregates affects primarily the quantity of water required in concrete, and as its influence on the tensile or bending strength, on the one hand, and on the compressive strength, on the other, is of the same nature, it is to be expected that the rules for granulation laid down with regard to compressive strength are also conducive to the production of high tensile or bending strengths. Fig. 1 illustrates the extreme Fuller curve values at present recognised in Germany for reinforced concrete mixtures. Comparing group 2, Table 1, with these, it will be found that the Fuller curves lying within the zone marked 'very good' do actually give the best bending strengths. Considering groups 2—4 and keeping an eye on the exponents x and y separately while doing so, it becomes clear that the most suitable percentage of sand in relation to the whole of the aggregates for concrete worked soft is between 50 and 60, also when the material is naturally graded. In group 3 it is conspicuous that sand whose granulation is unfavourable according to Fig. 1, influenced the bending strength less unfavourably than it did the compressive strength as long at is was not present in excessive quantities. It is possible that this result was affected by the fact that the finely granulated mortar dried more slowly. *Pfletschinger* (16) has found that for bending strength it is important to have the coarse aggregates well graded (> 7 mm), while this does not matter so much for compressive strength.

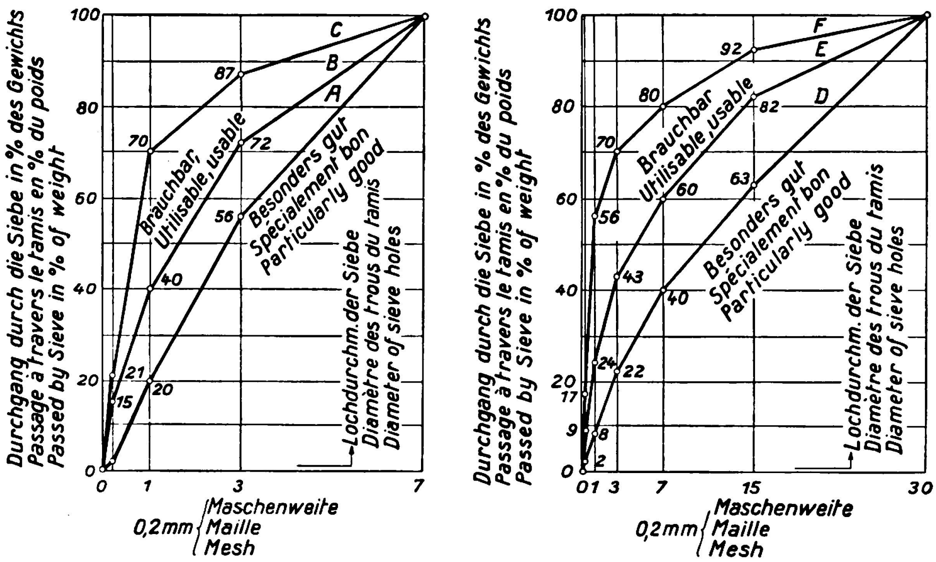

Fig. 1.

Ultimate Fuller Curves acc. to Germain Commission for Reinforced Concrete. Fuller Curves for Sand only, Fuller Curves for all Aggregates.

5) Shape of granules and surface character of aggregates.

The shape of the granules and the surface character of the aggregates determine the quantity of water required for fresh concrete — given a certain cement content and size of granules — for the attainment of a certain workability. In order to keep down the water content, it is better to have granules of as round and cubical a shape as possible (Length: breadth: thickness between 1:1:1 and 1:0.6:0.2, as laid down in the general specifications for surfacing on the German arterial roads), and not an over-rough surface. This applies particularly for soft and liquid concrete. Moreover, the adhesion of the cement to the stones and the cohesion between mortar and coarse aggregates is dependent upon the surface character of these aggregates. This factor would seem to matter more for tensile and bending strength than for compressive strength, for when the concrete is subjected to tensile or bend stressing the granules of the aggregate cannot mutually support each other. Aggregates with rough and irregular granule surfaces can therefore have an advantageous effect on the tensile and bending strength provided that the unfavourable effect of increased water content does not outweigh its good influence. Accordingly, it can be seen from groups 5—7, Table I, that the use of broken instead of naturally graded aggregates improves the relation of the tensile or bending strength to the compressive strength (increased values of x and y); it will also be observed, however, that by this procedure the absolute values of tensile and bending strengths are by no means always favourably affected. In group 6, for example, in the case of liquid concrete, there is even a perceptible lowering of the strengths owing to the broken aggregates. Groups 4, 5 and 7 show that as regards tensile and bending strength crushed sand and natural sand are of the same quality, but that crushed sand has quite a bad effect on the compressive strength. There is thus no cause to prefer crushed sand to natural sand in respect of tensile or bending strength.

6) Type of stone in aggregates.

The tensile strength of the stone used in concrete aggregates is generally greater than the strength of concrete usually attained till now. If it is remembered, however, that the tensile strength at present attainable in comparatively old concrete can be estimated at app. 55 kg/cm² on the basis of the bending strengths observed (2 p. 90), and that there are types of stone, suitable for use as concrete aggregates, but with a smaller tensile strength than this, then it becomes clear that the tensile strength of the stone must be considered if especially high qualities are to be attained (12 footnote 12). The bending strength of the stone is hardly of much importance for the strength of the concrete, for in the tensile and bending tests only particularly longshaped pieces can be destroyed by bending.

The surface character of the aggregates, the importance of which has already been discussed, depends upon the type of stone and — in the case of broken aggregates — the manner in which it was crushed. Nothing has yet been said of the extent to which the stone's capacity for absorbing water has to be considered, whether with the object of improving the water-cement ratio (provided that the aggregates were not wetted beforehand) (16), of reducing the rapidity of drying out, or perhaps even of increasing adhesion of cement to stone.

Aggregates composed of types of stone or artificial material which — as for instance blast-furnace slag or cement clinkers — react chemically in conjunction with the cement and thus produce stronger binding, can be employed to advantage.

A few instances of the influence of the surface character of various types of stone on the tensile and bending strength of concrete are given by *Dutron* (group 7, Table I) and *Guttmann* (3).

Finally, the type of stone has also to be considered for its effect on self-stresses (cf. S. 8).

7) Casting concrete.

The more compact concrete is made in casting, the greater is its tensile and bending strength likely to be. Thus *Graf* has repeatedly recorded bending strengths of up to 80 kg/cm² in machine-rammed concrete products — in individual cases as much as 120 kg/cm² (2 p. 90). Greater differences in the performance of rammed concrete, however, are only possible with moist mixtures. Vibration has a praticularly favourable effect in the case of the latter, not only because it affords greater compactness, but also because of the smaller water-cement ratio required. Group 8, Table I, shows the advantages of vibrating over ramming, and it should be noted that the amount of ramming work done was exceptionally great. The higher strengths given for the same cement content are those for vibrated, the lower those for rammed concrete. The relation of the bending to the compressive strength remained the same for both methods of compacting. Mortar which was sprayed on attained high tensile strength.

8) Moisture and temperature.

The tensile and bending strength of concrete is largely dependent on the effects of moisture and temperature. For as soon as the moisture or temperature is unevenly distributed over the section of the concrete body, stresses are set up in the concrete itself although there may be no forces acting from outside. These self-stresses form a pre-stressing of the concrete and cause the strength calculated from rupture loads to work out smaller than the real strength.

Differences in the degree of moisture in concrete, causing self-stressing, are set up when, for instance, moist concrete dries out or dry concrete is moistened, since the change in the degree of moisture and the consequent shrinking or swelling of the concrete takes place gradually, proceeding from the surface towards the interior of the body. Now if, for example, the surface region is drier than the core of the section, the former is restrained from shrinking to the extent corresponding to its degree of moisture, so that tensile stresses are set up in the surface region and balanced out by compressive stresses in the core (13 p. 106). When the concrete is moistened the condition of stressing is reversed.

Fig. 2, based on experiments carried out by *Graf* (19 Fig. 4), gives some insight into the actions set up by differences in degree of moisture during the drying process. Here bodies of various sizes of cross section but under the same conditions of storing were observed. On the assumption that in the smaller bodies, practically, there are no moisture differences within the section, the bold lines show to what extent the concrete on the surface of the larger bodies would have contracted if it had not been expanded by self-stresses and prevented by moisture from the interior of the section from drying out as quickly as the concrete of the smaller bodies. It will also be noted from the smaller axial contraction of the large bodies (dotted line), how much more slowly these latter dry out than the smaller bodies; in this connection it should also be considered that compressive self-stresses increase the axial contraction above the amount caused by shrinkage alone. And, finally, a comparison between the surface and the axial contraction of the large bodies (dotted and bold line) shows to what a great extent the differences in contraction to be expected from the differences in degree of moisture are cancelled out by self-stresses. The latter become correspondingly greater as the differences in degree of moisture in the section increase and the more the cement and aggregates tend to shrink or swell. The differences in degree of moisture are dependent upon the relation between surface and section of the body, upon the character of the concrete pores which determine the rapidity with which the drying out process proceeds towards the centre (20 Pt. I) (21) and upon the rapidity with which the surface region dries out; this speed is higher, the greater the difference between the moisture content of the concrete and that of its surroundings (22 p. 136). To retard drying, the concrete is best coated with an isolating substance (3) (23)

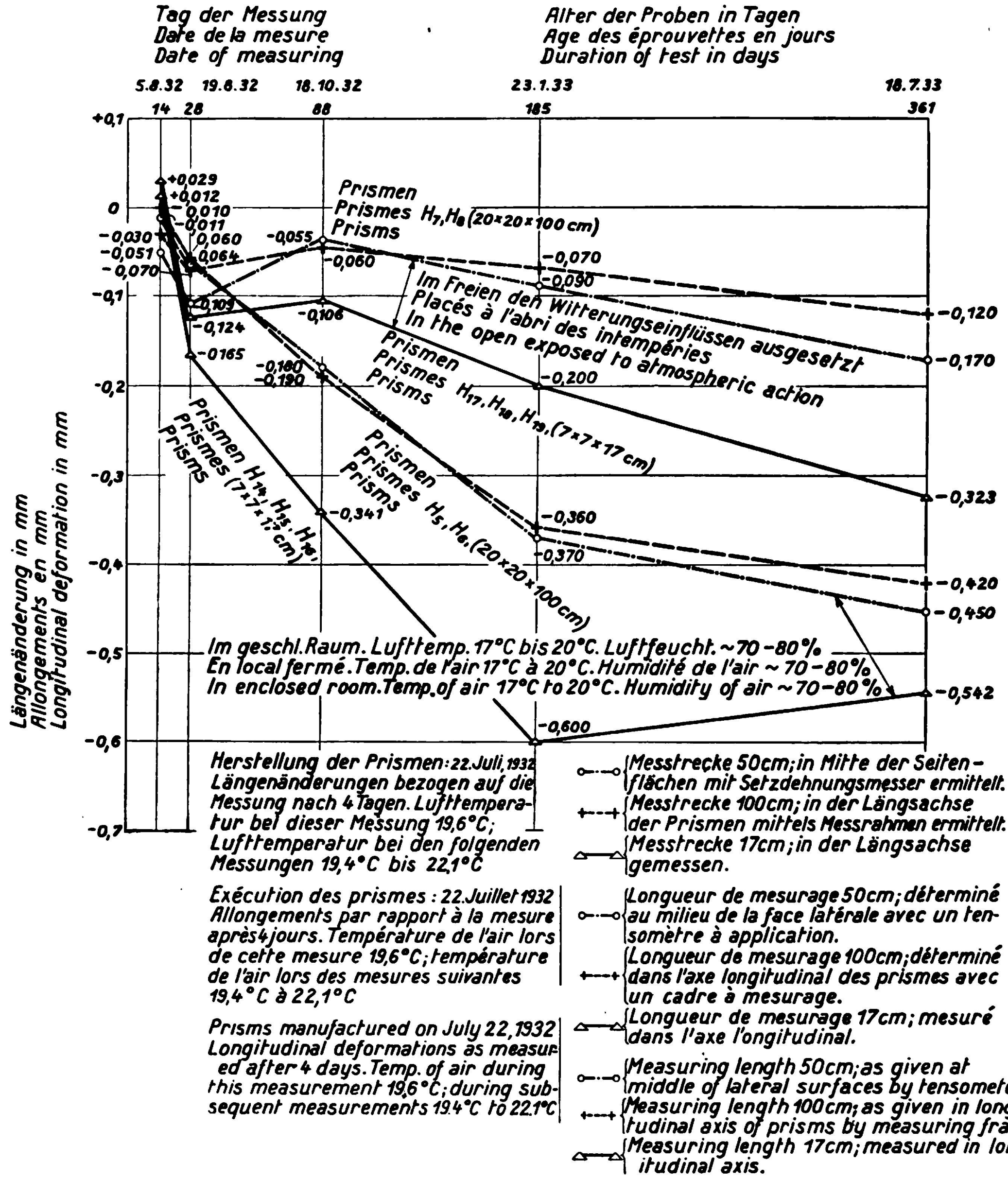

Fig. 2.

Shrinking and Swelling of various Sizes of Bodies.

(39 p. 139). The self-stresses, on the other hand, decline as the degree of elasticity decreases and as the creeping of the concrete increases (24). Furthermore, the rapidity of hardening has to be taken into account. The quicker the concrete hardens, the sooner does the magnitude of the self-stresses fall behind that of the real strength, whereas the self-stresses increase in quicker-drying concrete because the degree of elasticity grows more rapidly and creeping

stops sooner than in slow-drying concrete. These numerous influences, some of which have mutually contrary action, result in differences in storing conditions having variously marked effects on the tensile and bending strengths of concretes of various compositions (1 p. 90 and 94) (12 Table 12).

Although a decrease of tensile strength is always to be expected when the degree of moisture alters, *Graf* found that, on moistening concrete which had been dried for a more or less long period, the bending strength increased (26 Table 9). The reason for this can be found in the fact that the self-stresses act throughout the whole section during the tensile test, so that tensile stresses in the core decrease the tensile strength when the surface region is moistened, whereas in the bending test only the self-stresses in the surface region are decisive. The latter, however, are at first compressive stresses when moistening takes place, and therefore increase bending strength. It is worth noting that in these tests the tensile stresses in the core acting vertically to the direction of compression decrease the compressive strength.

In addition to the self-stresses described, long storage under water can also decrease the tensile and bending strength of concrete (1 p. 90) (26 Table 3).

Warming and cooling produce self-stresses of the same kind as do moistening and drying; the magnitude of these stresses naturally depends upon the differences of temperature in the section (23).

On summing up these considerations it will be found that the most unfavourable effect on the tensile and bending strength is produced by abrupt changes of storing conditions. On the other hand, a beneficial action can be created if the surface of the concrete is kept moist for a considerable period and then allowed to dry as slowly as possible (4 p. 49).

9) Age.

The tensile and bending strength of concrete grow with its age in accordance with the rapidity with which the cement hardens. This fact, however, is partially or wholly cancelled out by the effects of storage as elucidated in S. 8, so that, according to circumstances, over a considerable period the tensile and bending strength can be observed to remain constant (27 p. 51) or even to decrease (12 Table 12) (26 Table 3), even though the compressive strength keeps on increasing. In this connection it should be noted that in general concrete dries very slowly (1 p. 89), much more slowly than it absorbs water (28 Fig. 24) (22 p. 140).

If, after the storage conditions have been altered, the tensile and bending strength again increase with progressive equalisation of the degree of moisture, owing to the corresponding decrease of self-stresses, this increase is often greater within a certain period than is the simultaneous increase in compressive strength. At the end of this period the exponents x and y are therefore often greater than at the beginning (cf. Table 2) and approach the values set up by a hardening process involving no self-stresses (cf. the two last lines of the Table). This Table shows further that concrete containing more water hardens at a different rate than does concrete containing little water, so that the self-stresses increase and decrease at different rates in accordance with the water-cement ratio.

Table 2.
Influence of curing and age on the tonsile strength of concrete [1].

Storage	Age	$W_1 = 0{,}53$			$W_2 = 0{,}61$		
		K_{z_1} kg/cm²	K_{d_1} kg/cm²	y_1	K_{z_2} kg/cm²	K_{d_2} kg/cm²	y_2
1	2	3	4	5	6	7	8
7 days moist then dry	28 days	12,4	225	0,466	12,0	191	0,474
	45 days	13,7	253	0,472	11,8	209	0,463
	6 month	19,5	337	0,511	15,3	297	0,480
	1 year	23,7	371	0,536	23,1	329	0,543
Continually moist	45 days	19,0	224	0,545	17,0	201	0,534

10) Alternate loading.

Tensile and bending strength is decreased by frequently repeated loading and unloading. In terms of surge-load strength, the permanent bending strength works out at about half the bending strength as determined in the usual manner (29 p. 117). In this connection it should be noted that frequent changes of temperature also act as alternate loading.

d) Means of increasing the tensile strength of concrete.

Summing up the preceding considerations, the following means and measures can be recommended for increasing the tensile strength of concrete:

1) First of all, a suitable brand of cement should be selected. The cement should produce maximum bending strength as determined by the testing process with soft mortar of various granule sizes; it should also shrink as little as possible and enable the concrete to be worked easily even when the water content is small. Slow-hardening cements should be given preference provided they attain sufficient strength and allow of good curing of the concrete.

2) Aggregates should be used whose tensile strength is greater than that of the desired tensile strength of the concrete. The best materials in this connection are those which show little shrinkage, vigorous creep and a low coefficient of elasticity. It is advantageous to use rough-surfaced granules provided that the amount of water required is not too greatly increased. Attention must be paid to this point when using crushed stone sand and stone chips. When applying the stone, care should be taken that no fissured pieces are let through.

3) For grading the aggregates the same general principles hold good as those elaborated for the production of concrete with a maximum of compressive strength. It would, however, seem advisable to use at least 50% sand (granulation < 7 mm, calculated on the total weight of the aggregates, even for naturally graded materials. For coarse aggregates care should also be taken that the granules are well graded.

5) The quantity of water in concrete, i. e. water-cement ratio, should be kept as low as possible. It may thus be practicable to use liquid substances for wetting.

6) Concrete should be compacted as much as possible. Provided the concrete is thick enough, vibration will therefore be of advantage.

7) Concrete should be kept moist as long as possible at the beginning and dried as slowly as is practicable. Repeated and particularly abrupt changes of the moisture and temperature conditions to which the concrete is exposed should be avoided, especially while the concrete is still fresh.

II. Ductility of concrete.

a) Ductility of concrete under loads of brief duration.

Tests made to ascertain the ductility of concrete under tensile and bend stressing yielded the following results: — Large test pieces undergo for the same stressing somewhat greater elongations as smaller bodies (30) (31).

Greater rupture elongations are recorded for bending tests than for tensile tests (25 p. 39); the reason will be found under 1 a. The modulus of elongation $\alpha = \frac{1}{E}$, calculated on the whole or elastic deformations, coincides for small stressing in tension and in compression. For greater but equal stresses α becomes a little larger for tensile than for compressive loading (30 p. 50). The modulus of elongation α increases with the loading of the concrete.

In the case of concrete composed of the same materials (brand of cement, type of stone in the aggregate), the modulus α decreases for the same stressing, the greater the strength of the concrete (30 p. 50) (24) (31). By changing the brand of cement or the type of stone, thus varying the deformability of the aggregates, concrete of various degrees of ductility can be obtained for the same strength. In Table 3 three ratio values taken from *Hummel*'s investigations (24) are quoted, which show to what extent the ductility of concrete can be influenced in this manner. It is noteworthy that the greater ductility of concrete becomes the more pronounced, the more the stressing approaches the rupture stress point.

Table 3.

Ratio Values of Coefficients of Elongation $\alpha = \frac{1}{E}$ after Hummel.[24]

Concrete distinguished by:	K_d kg/cm²	K_b kg/cm²	15	25	$\sigma_{bz} =$ 35 kg/cm²	40	45	K_b
Type of stone in aggregates	555	48	1	1	1	1	1	1
	510	49	1,06	1,02	1,04	1,07	1,22	1,35
	479	48	1,35	1,32	1,33	1,34	1,42	1,53
Cement	532	48	1	1	1	1	1	1
	544	48	1,0	0,98	1,0	1,02	1,05	1,25
	500	47	1,0	1,08	1,24	1,55	—	1,95

The ductility of concrete is evidently influenced by self-stresses, according as these are set up by the storage conditions. However, the experiments carried out (24) (30) (32) cannot be adequately compared to yield general conclusions.

To what extent the coefficient of elongation α is affected for tensile and bend stressing of the concrete by frequently repeated loading and unloading below the permanent strength limit, does not become apparent.

b) Ductility of concrete under lasting stationary tensile loading (creeping properties).

The creeping of concrete under tensile stress has hitherto been but little investigated. Only one test is known — reported by *Glanville* (33) — in which the degree of creepage for concrete, tested at an age of one month, was the same for tensile stressing as for compressive loading. In the test reported it amounted 0.1 mm/m after six months under a stress of 10 kg/cm². As creepage increases in direct ratio to stress, it will become greater under loading in the vicinity of the ultimate tensile strength than does the rupture-point elongation in the loading test of short duration, i. e. app. 0.0045 mm/m per kg/cm² tensile strength (4 p. 51). If it is permitted to generalise as regards the observations made on the creep ratio for tensile and compressive stressing, then investigations carried out into the creeping of concrete under compressive stress [according to a report on tests made by *Davis, Glanville,* etc. (34)] show that the ductility of concrete under loading of long duration may be very much greater than the ductility recorded in short-period tests, and that it can also be influenced to a much greater extent by the composition and treatment of the concrete.

c) Importance of the ductility of concrete in the formation of cracks.

Distinction must be made between rupture-point elongation and the coefficient of elongation α.

The magnitude of the rupture-point elongation is of no importance in all cases where the carrying capacity of the structure is exhausted as soon as cracks appear. Here the only thing that matters is that the tensile strength of the concrete is sufficient to take up the stresses with safety. In all other cases the danger of cracking becomes less, the greater the rupture-point elongation of the concrete. In this connection it is not always insignificant whether the greater rupture-point elongation corresponds to a greater or a smaller tensile strength. Take, for example, a concrete roadway slab which is expanded by friction forces while drying. The following facts will be observed: — The magnitude of the friction forces is limited. As soon as its limit is reached, the slab begins to slip on its foundation and does not undergo further expansion. The greater the forces necessary to expand the slab to rupture-point, i. e. the greater the tensile strength of the slab, the greater is the likelihood of the slab slipping and the avoidance of cracks.

The magnitude of the coefficient of elongation α has an indirect influence on the formation of cracks. The greater α is, i. e. the more ductile the concrete is, the smaller will be the stresses set up when deformation of the concrete caused by changes in temperature or degree of moisture is restrained (cf. I 8 also). However, the smaller these stresses are, the less will be the danger of their exceeding the tensile strength either alone or in conjunction with the stresses set up by loads.

The creeping power of concrete acts in the same sense as the coefficient of elongation α. Creeping primarily decreases the shrinkage stresses, which develop very slowly and whose action covers a long period (23) (24) (34).

Summing up, it will be seen that maximum attainable ductility of the concrete combined with greatest possible tensile strength is desirable if the possibility of crack formation is to be reduced. This statement, however, requires some qualification. As greater ductility of the concrete under tensile and compressive stressing also corresponds in general to a greater malleability under compressive stressing, such great deformations can take place in structural members subjected to bending as the deformability of the concrete increases — particularly because of creeping — that a far-reaching change occurs in the distribution of stresses as calculated in the usual manner, and the degree of stability and safety against cracking is reduced (34) (35).

III. Formation of cracks in reinforced concrete.

a) General.

In reinforced concrete construction the admissible elongations of steel, $\varepsilon_e \geq 0.6$ mm/m correspond to admissible stresses on the steel of $\sigma_e \geq 1200$ kg/cm^2, while the maximum rupture-point expansion of concrete for tensile tests attained up to the present amounts to about 0.2 mm/m (36 p. 3) — for bending tests not more than 0.3 mm/m (24). For this reason cracks generally occur in reinforced concrete structural members well below the safe load limit. Experience has shown that these cracks do not endanger the actual life of the structure as long as they do not become so wide that the steel is exposed to destructive influences (11, Report by *Krüger*) (37) (38). Measures for reducing the possibility of crack formation must therefore aim at

1) restricting the actual occurrence, i. e. taking care that expansions which are too great for the steel to withstand are confined to as small a portion of the structure as possible, and
2) preventing the gaping of unavoidable cracks.

b) Initial stresses in reinforced concrete.

As is well known, self-stresses are set up in the concrete of reinforced structural members resting on movable supports by the slippage resistance of the steel reinforcement as the concrete shrinks and swells. When shrinking occurs these self-stresses are tensile, while swelling produces compressive stresses, corresponding in the steel to compressive and tensile stresses respectively. These stresses in concrete and steel are together designated initial stresses because they are already present in structures not yet subjected to loading.

The magnitude of these initial stresses is difficult to obtain from tests, for the following reasons (39 p. 127) (40) (33): The contractions of the concrete in shrinking are transmitted to the steel reinforcement by the slippage resistance of the latter. The resistance itself is set up by the friction of the steel against the concrete and by gripping forces caused by the special constriction of the concrete (36 p. 32). Slippage resistance develops but gradually as the concrete hardens. It is therefore likely that in the first stages of hardening movements are possible between concrete and steel without the creation of stresses. Subsequently the shrinkage contraction of the concrete is transmitted from the end of the test piece to the reinforcing steel by the slippage resistance, whereby the relative movements gradually decrease until, in the middle region of the body, they disappear entirely. In like manner the initial stresses increase from 0 at the end to their maximum value at the middle zone of inertia. The law governing this increase, and consequently the length of the zone of inertia also — which does not appear at all in short bodies — are unknown. As soon as stresses are set up in concrete, it begins to creep. It is therefore impossible to arrive at the initial stresses in concrete from the difference in contraction of reinforced concrete as against that of shrinking concrete. The elongation of the concrete caused by the reinforcing bars, as against the contraction in the case of unrestrained shrinkage, is, on the contrary considerably greater owing to creeping than the elastic elongation due to initial stress (25 p. 36). The magnitude of the initial stress can thus only be measured by measuring the malleability of the steel in the zone of inertia. A test of this kind was carried out by *Glanville* (36 p. 53). The result, however, cannot be generalized owing to the size of the test piece. Initial stresses are not evenly distributed over the concrete section. In close proximity to the steel, creeping is greatly assisted by slippage resistance. This influence of the steel decreases as the distance from it increases, at first rapidly, then more slowly. For this reason test pieces showed a distinct curvature of the end surfaces (25 p. 37).

It is at any rate true of the zone of inertia that the contraction of the concrete, besides that of the steel reinforcement caused by the contraction of the concrete, in the case of unrestrained shrinkage δ reduced by the elastic elongation $\frac{\sigma_b}{E_b}$, and the creepage $\varkappa$ of the concrete, must be equal to the contraction of the steel $\frac{\sigma_e}{E_e}$. Here the creepage is a function of the temporal course of shrinkage and of the magnitude of the stress σ_b. Moreover, there must be equilibrium in the section. From the two equations: —

$$\delta - \varkappa - \frac{\sigma_b}{E_b} = \frac{\sigma_e}{E_e} \quad \text{and} \quad \sigma_b F_b = \sigma_e F_e$$

we obtain with $\mu = \frac{F_e}{F_b}$

$$\sigma_b = \frac{(\delta - k)}{\left(\frac{E_e}{E_b} + \frac{1}{\mu}\right)} \cdot E_e.$$

The initial stress in concrete therefore becomes the greater, the more the concrete shrinks and the higher the coefficient of elasticity of the concrete and the percentage of reinforcement are; whereas it decreases, the more the concrete creeps. In this connection it is mainly a question of the creeping that takes place during the initial stages of hardening; this initial creeping may be considerable when the hardening process is sufficiently slow, even though the creepage when the concrete is older remains small, as is desired when allowing for permanent deformation under loads.

The danger of cracking is increased by initial tensile stresses in concrete. If the formation of cracks due to intial stresses is to be reduced, concrete of little shrinkage, high ductility and slow rate of drying should therefore be used, and care taken that a slow, steady drying rate is maintained (cf. I 8). In other words, the same precautions have to be taken as have already been recommended for unreinforced concrete. In addition, the percentage of reinforcement should be kept as low as possible. For this reason, too, weld splicing of the steel bars is more practical than overlapping or turn-buckles.

From the observations made concerning the fading of the action of steel on concrete in the proximity of the reinforcing bars, it can be concluded that for the same cross-sectional area of concrete and steel the deformations of the concrete, set up by the steel, cover an increasing port of. the sectional surface, the more evenly the section of the steel is distributed over the section of the concrete, i. e. the more numerous the bars with a correspondingly smaller sectional area of the individual bar. The range of elongations in a concrete section is therefore reduced by better distribution of the steel section, and it is probable that the initial stresses are also decreased thereby.

Initial stresses, too, are most likely the reason why in high-webbed beams with a strongly reinforced tensile zone — and especially in I-shaped beams, the cracks appear first in the portion of the web situated above the reinforcement. As explained above, during the drying process the concrete has been expanded in the proximity of the steel bars, so that the contraction of the reinforced zone has kept much lower than for unrestrained shrinkage. Owing to cohesion the concrete in the web must now expand just as much above the reinforced zone, and this is also only possible by creeping. Here, however, there is no slippage resistance from the reinforcement to encourage creeping, so that the amount of creepage becomes smaller as the distance from the reinforced zone increases and a larger portion of the elongation to which the concrete is subjected produces stresses. The initial stresses in the web above the reinforced zone are thus greater than in the zone itself. This phenomenon is particularly marked in I-shaped sections, because between the wider tensile zone and the narrow web greater moisture differences and therefore greater shrinkage differences can easily occur. To protect the web against cracks caused in this manner it is advidable to attain good distribution of the reinforcement near the surface — a measure which has already been frequently been employed (e. g. 41), even if only in consideration of the fact that above the reinforced zone, too, the concrete is subjected to high tensile load-stresses (42).

c) *Development and consequences of cracks in reinforced concrete.*

Emperger (36) recently made an exhaustive investigation into the occurrences that take place in the neighbourhood of a crack. His observations yielded fundamental coincidence with the occurrences apparent from, or indirectly indicated by, the results of tests carried out with the object of determining initial stresses (cf. III b). Instead of creeping under the protracted influence of loading, when cracking takes place, plastic elongation is caused by the effect of slippage resistance in the direct proximity of the reinforcement; this is greater than the normal elongation (36 p. 18), and decreases rapidly as the distance from the reinforcement increases (25 p. 40).

The tensile force of the steel at the point of cracking must again be transmitted to the concrete by the slippage resistance. First of all there will be an area, situated just beside the crack, in which the concrete has loosed itself from the steel owing to its inability to follow the elongations and cross-sectional constriction of the steel (zone of separation); then follows a zone in which the concrete undergoes plastic elongation in progressively increasing layers and where the slippage resistance, which rapidly attains its maximum value, operates (plastic zone). Here the concrete assumes part of the tensile force from the steel, until finally the elongations of the latter no longer exceed the elastic ductility of the concrete (elastic zone). (36 Fig. 20) (25 p. 53). As the stressing of the steel increases and the distance between the cracks decreases, the two last-mentioned zones vanish successively, so that eventually the steel slips along its whole length between two cracks.

It can be concluded from the occurrences in the proximity of a crack that the co-operation of the concrete in the tensile zone is afforded correspondingly longer, the greater the slippage resistance of the reinforcement and the plastic deformability of the concrete. As far as the concrete is concerned, the slippage resistance increases with the strength of the concrete, though to a lesser degree (25 p. 56), whereas the plastic deformability increases as the strength of the concrete decreases (36 p. 73 (30 p. 50). Slippage resistance can be more easily influenced by the type of reinforcing bar used, e. g. steel with as rough a surface as possible, or rods of special cross section, such as bulb or twisted bars, whose shape affords good connection with the concrete (25 p. 58) (36 p. 73). The co-operation of the concrete can also be made more effective by causing a greater cross-sectional area of the concrete to be affected by the reinforcement, e. g. by better distribution of the cross-sectional surface of the steel (25 p. 41) or by the arrangement of reinforcement on all sides (cross reinforcement, wire mesh, expanded metal). In this connection it should be noted that cross reinforcement (stirrups) placed between the main reinforcement and the surface of the concrete favours the formation of cracks.

More active co-operation of the concrete in the tensile zone has the following effects: — The mean value of the stress to which the steel is subjected under a certain loading is reduced, and with it the elongation of the tensile area (25 p. 51). Further, the distance between the cracks becomes smaller, since the unloading of the concrete from the first crack only operates over a small distance (25 p. 48). However, the more numerous the cracks are, the less they will gape (25 p. 50). The reason for this is the fact that the expansion of the steel along a certain length in which cracks have occurred, is chiefly dependent on the elongation of the steel in the cross sections of the cracks, since there the steel is stressed most. Now, the more numerous the cracks, the smaller will be the fraction of the total elongation of the steel pertaining to the separate cracks.

As the stressing of the steel increases, the cracks gape more widely; unless, of course, new cracks appear, before the slippage resistance between the existing cracks has not been overcome. As soon as the slippage resistance is exhausted, the width of the cracks ceases to be dependent upon their number (25 p. 51). On the contrary, some of the cracks may gape exceptionally wide, while others may close again somewhat. Thus tests with frequently repeated loading and unloading (36 p. 114) (43) revealed that, in the case of round steel reinforcement, bars whose slippage resistance has been overcome by permanent loading, the greatest width of crack increased considerably with repetition of loading, although the number of cracks remained the same. On the other hand, where Isteg steel was used and slipping in the concrete was impossible, this width remained unchanged. When such permanent loading is applied the number of cracks at first increases but soon enters a state of inertia. New cracks can also be formed under permanent, stationary loading as a consequence of creeping, since the neutral axis is displaced towards the side under tension and the stressing of the tensile area thereby increases (33) (34).

The gaping of cracks is greatly affected by the fact that the steel has suddenly to take over the force formerly carried by the concrete up to the moment of cracking (36 p. 44) (44). The bars are therefore subjected to additional elongation at the spot where cracking occurs — the only place where they can expand without restraint. The cracks will thus gape the wider, the smaller the relation between the cross section of the reinforcement and that of the ruptured concrete, and the greater the former tensile force of the concrete, or, speaking generally, the greater the width of the tensile area (44) and the tensile strength of the concrete. When the crack forms, the concrete springs back in consequence of the unloading. This movement is smaller in the proximity of the reinforcement, owing to slippage resistance, than at a distance from it where the elastic elongation was also greater. The crack therefore gapes somewhat more at the surface of the body than near the steel bars (36 p. 48), just as it begins at the surface and progresses in the direction of the reinforcement (39 p. 117).

It is a fact of particular importance that the cracks remain thin under dead weight and therefore close again as far as they can when traffic load has been removed (45). The permanent width of crack seems to be chiefly dependent upon the permanent elongations of the concrete above the reinforced area. In the crack itself, too, permanent elongations of the steel occur, since the bars are prevented by slippage resistance from springing back completely (36 p. 73). There is as yet no information available as to the width of crack that can be allowed to remain without ever endangering the member. Investigations with centrifugal concrete, i. e. with concrete of a high degree of compactness (37), gave a permissible permanent crack-width of up to 0.3 mm, and a temporary width of 0.5 mm.

Gehler (44) concludes from the crack-widths measured in T-beams at a stressing of steel of 1200 kg/cm^2, that a width of $^1/_8$ mm is permissible. *Graf* (43) comes to a similar result. The object of such definite conclusions is to form a standard by which to judge the results of tests as to the admissibility of increased stressing in the steel, independent of the quality of cement and the type and arrangement of the reinforcement. It is, however, doubtful whether, with the knowledge and experience at present available, the width of cracks can be restricted to a certain limit (cf. 44 Table III).

Our discussions on the gaping of cracks show that the degree of safety against cracking, calculated as the relation of the loading at which the crack occurs to the permissible loading, or as the relation of the stressing of steel, calculated in the usual manner without considering the co-operation of the concrete, to the permissible stressing of steel (44), in general only attains a magnitude of practical utility when it is greater than 1. As long as it keeps below 1, an increase does afford advantages if, in consequence of greater safety against cracking, cracks subsequently created under dead weights gape less than cracks which appeared prematurely.

The safety against cracking, as has been laid down in the foregoing, increases with the bending strength and ductility of concrete, the efficiency in type and arrangement of the reinforcement, and the difference between the calculated stressing of the steel and the real stressing due to the co-operation of the concrete and finally, inversely as the reinforcement ration $\mu = \frac{F_e}{bh}$ (25 p. 24) (45). It is therefore greater for slabs with crosswise reinforcement, supported on all sides, than for mono-axially stressed slabs, and for slabs than for T-beams; in the case of the latter, it increases with the width of the tensile area of the concrete (44).

d) Measures for the prevention of cracking in concrete.

From Sections b) and c), III, a summary reveals the fact that if the formation of cracks in reinforced concrete is to be reduced, concrete that shrinks little, is very ductile and hardens as slowly as is practically possible should be employed, and further that the drying process should take place slowly and steadily. As long as there is any possibility of preventing cracking under safe loads, it is a question of attaining a maximum degree of tensile strength in the concrete. If cracks are unavoidable, however, as for instance in T-beams under customary conditions, then the ductility of the concrete becomes a more important factor than its tensile strength.

Particular care should be taken that in the tensile area the co-operation between concrete and steel is sustained up to the highest possible point of elongation of the steel. For this reason the cross section of the latter should be distributed over as large an area as possible. Rough-surfaced steel bars should be given preference, together with such as possess high slippage resistance, like bulbed or twisted rods. (Naturally these bars must also possess the other properties required in reinforcing material, and their shape should not be such as to cause splitting of the concrete.) Transverse reinforcement, rigidly connected with the tensile reinforcing at suitable places, also offers advantages.

Besides these generally applicable measures, it is possible in special cases to cause such great compressive pre-stresses in the concrete by pre-stressing the reinforcement, that no tensile stresses whatever occur in the concrete under safe loads. This possibility was investigated quite a long time ago (25 p. 44), but it remained for *Freyssinet* (cf. work just mentioned) to succeed in exploiting it after he had recognized the significance of creeping in the amount of pre-stressing required and found new ways of reducing volume deformations in concrete (20). His views and elucidations on the subject of the extent to which the properties of concrete can be influenced will probably contribute also to the clearing up of the questions treated in the present report.

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Summary.

The present paper is primarily an exploitation of the copious literature dealing with the theme under discussion. It may be summarized as follows: —

The tensile strength of concrete is mainly dependent upon the tensile strength of the respective cement used, calculated as bending strength on the more recently introduced testing methods, with plastic mortars of mixed granule sizes. In general, the same measures should be taken for attaining good composition as for producing a concrete of maximum compressive strength. Broken aggregates improve the relation of the tensile to the compressive strength, but in general do not produce greater tensile strength than do naturally graded aggregates when the concrete is soft and liquid. This is due to the greater quantity of water needed. Tensile strength can be influenced to a greater extent by the properties of the stone aggregates than compressive strength can. Slow drying out of the concrete is a particularly important factor.

In evaluating the separate influences it proved practicable to follow the relation of tensile to compressive strength with the aid of exponents of compressive strength which allow for the fact that this relation depends upon the magnitude of the compressive strength.

Since the ductility of concrete is an important factor in the formation of cracks, the effect of this property was also investigated. It generally decreases as the strength increases, but can be influenced by the type of cement and type of stone aggregates employed. Creeping has a beneficial effect on tensile strength, since it reduces undesired shrinkage stresses.

Tensile strength is only of supreme importance as regards the prevention of cracks in concrete when there is a possibility of completely avoiding the formation of cracks under safe loads. It is only in this case, too, that high degree of safety against cracks, i. e. the relation of the cracking-point load to the admissible load, is of practical importance. Where cracks cannot be avoided, the ductility of the concrete is more important than its tensile strength, and slight gaping of cracks more important than if these cracks develop only after great loads have been applied.

For the rest, it is a question of dividing up the reinforcement as much as possible and of using types of reinforcement which develop great slippage resistance in the concrete. When treating the subject of crack formation it was proved that the compound action of concrete and steel when initial stresses are set up coincides in principle with the occurrences in the proximity of a crack, and that the initial stresses are probably smaller than has hitherto been assumed, owing to the creeping of the concrete.

IIb 2

The Tensile Strength of Stressed Parts in Reinforced Concrete

Zugfestigkeit des Betons in Eisenbetonkonstruktionen.

Sur la résistance des pièces tendues dans les constructions en béton armé.

G. Colonnetti,
Professeur à l'Ecole Supérieure d'ingénieurs de Turin.

All those who have carried out experiments agree that, given the same class of metal section, tensile strength increases in reinforced structures and cracks are reduced in proportion as the number of bars used is increased, and therefore the diameter of these latter is reduced.

Meanwhile, even though this fact appears to have been definitely laid down and confirmed by experiment, the interpretation given to it by various authors appears less definite.

It is no use referring to circumstances which are evident, such for instance, as the fact that the diameter of the steel bars decreases the ratio between the circumference of their cross section and their area, however, if the conditions of adhesion between steel and concrete are improved — especially when the experimental determination made extends to cases in which girders are subjected to ordinary bending stresses in which in theory the adhesion would not even have any reason for coming into play.

As a matter of fact, that reference acquires some value, even a very definite and clear value as we shall soon see, only if, when trying to analyse what exactly takes place in a structure which has been concreted to increase its tensile strength, we set aside all those very elementary and often contradictory conceptions to which we have to resort when making static calculations.

Every one knows that in those calculations we set aside all participation of the concrete in resisting tensile stress, admitting that it is borne solely and entirely by the steel, if those calculations aim only at confirming the strength of the material: on the other hand, however, we rely entirely on its participation and admit that the internal stresses are distributed between the steel and concrete according to the respective moduli of elasticity every time we set about calculating deformation, no matter whether this is of direct interest to us or intended to be used when determining certain unknown hyperstatical quantities.

The truth is — and we are well aware of it — that in practice, neither the one nor the other case will arise; or to be more precise, the two hypotheses occur only exceptionally and then for certain metal sections; they pass then gradually from one to the other limit-case across a whole scale of intermediate static conditions in which the concrete is subjected to a part, but a part only, of the stress which really belongs to it.

It would be idle to try and introduce this partial participation of the concrete in the strength of the structure into those calculations, because such participation changes considerably according to circumstances, and in each particular case according to each part of the structure, because the degree of homogeneity of the concrete and its degree of adhesion to the steel vary, and above all because the number and position of certain very small, almost imperceptible cracks may be due to so many different causes.

It has been said that no reinforced concrete structure exists in which, after careful investigation, some of these imperceptible cracks will not be found, and as a rule they are the result of internal stresses set up in the structure when the concrete contracts, or resulting from variations of temperature.

In any case, where such cracks do occur, the resulting tensile stresses will all have to be borne by the steel. But in the adjacent zones, where the steel is surrounded by a sound, compact and thoroughly adhesive mass of concrete, the latter, forced to follow the course of distortion, will take an active part in the resistance and relieve the steel of a certain part of the strain which, in relation to the cracks, it will bear.

Now it is just in these alternating changes when the stresses pass to and fro between steel and concrete — changes that are unexpected and cannot be foreseen by static calculations — that tangential stresses are set up in the mass of the concrete, and these tangential stresses have nothing to do with those stresses which are connected with any shearing tension that may be present.

It is just these stresses which, if they exceed the limits of resistance of the material, may lead to the spread of the existing cracks or form new ones.

Meanwhile the problem to be solved is that of inducing a more rapid and effective participation of the sound mass of concrete in resisting stress in the structure — while limiting as far as possible the zones of low resistance — and preventing the tangential stresses from exceeding those limits or from reducing stability of the whole system.

* * *

But there is another point in connection with the theoretical presentation of facts which deserves to be closely and critically examined.

It is a well-known fact that one of the fundamental postulates on which the usual static theory of reinforced concrete structures rests is that the distribution of internal stresses does not depend on the particular methods of application of the external stress.

In view of the stresses belonging to a given section of the structure, it will be admitted that *De Saint Venant* was right when he said that the law according to which the internal strain is in the section itself is the only law and a very definite law, no matter how the forces which determine that strain are applied.

Now in reality the position is such that this method of application exercises an influence which must be taken into account, and which in this special case of reinforced concrete girders may even become very marked on account of the varying conditions under which, with reference to the loads actually applied, the concrete mass and the respective metal framework are placed.

As a matter of fact, cases in which external stresses, when being set up, are distributed between steel and concrete in such proportions as to result in distortion along the respective surfaces in contact with each other are so rare as to justify the hypothesis of the maintenance of plane sections.

It is much more likely that cases will occur in which the stress on a certain girder will affect the metal framework across the braces which are suitably arranged to connect the various steel parts. In that case the metal framework, subjected to distortion by the action of the stress, will also force the mass of concrete of which it forms part to become distorted and induce it to participate more or less actively in resisting the stress. But it is obvious that this transmission of stress from steel to concrete cannot take place except as a result of adhesion and the setting up of a system of tangential strain which is not justified by the stresses alone but by the particular method in which they are applied.

The opposite is likely to occur more frequently: the external forces which set up stress are generally applied in the form of superficial pressure bearing on the mass of concrete. Then it is the concrete which, subjected to deformation, causes the enclosed steel to become distorted and subjected to a fraction of internal stress, relieving the overstressed concrete to a greater degree than theory would seem to warrant. Once again the transmission of stress from concrete to steel cannot take place without a certain system of tangential strain which the stresses alone cannot account for being set up, und its cause must be sought exclusively in the fact that the state of equilibrium is not such as is laid down in theory.

This maintains its whole value of limit-theory which is to be confirmed in those sections of the girder that are sufficiently distant from the points of application of the external stresses. This therefore means that, under the ordinary conditions of loading girders in an ordinary reinforced concrete structure, the theory would never be rigorously confirmed.

It is not permissible, therefore, to disregard in practice the fact that under the conditions of load specified above, the internal tension in the concrete in the neighbourhood of the points of application of the load may assume, and actually do assume, higher values than those theoretically laid down, and these are indeed so much higher and extend more widely over the girders in proportion as the passage of stresses between concrete and steel proceeds more slowly.

And thus it is that — in other circumstances and in quite a new form — the same problem arises once more: namely, the problem of whether, and in what manner, this alternating passage of stresses can be accelerated and made more effective — without the resultant tangential tensions exceeding the limits of the resistance of the materials — and in this way the zones in which the abnormal distribution of the stresses occurs are reduced and the differences between such distribution and that laid down mathematically become less marked.

* * *

And thus, by going back to well-known and very elementary methods of calculation and adapting them to the present case, a problem of this kind might be brought nearer to solution.

Let us suppose, in order to crystallise thought, that a small steel disc with a diameter that we shall call $2\,r$, has one of its perpendicular cross sections stressed by a normal tension σ_f only, and in an adjacent section at a distance d_z from the preceding one, by a similar unique tension $\sigma_f + d\sigma_f$ (Fig. 1).

The equilibrium of the portion of steel between these two sections evidently requires that a tangential tension be exerted on its lateral cylindrical surface (this is possible owing to the adhesion of the concrete). This average sole tension τ must comply with the following formula, namely:

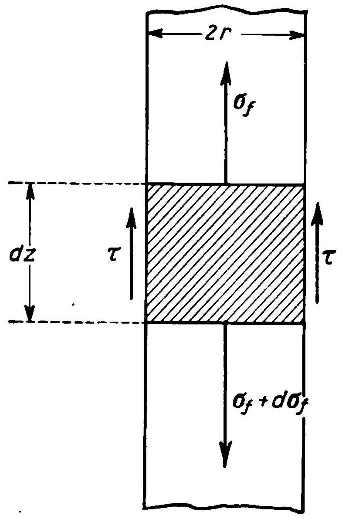
$$d\sigma_f \cdot \pi r^2 = \tau \cdot 2\,\pi r \cdot d_z$$

from which it is seen that:

$$\frac{d\sigma_f}{d_z} = 2\,\frac{\tau}{r} \tag{1}$$

And now let uns consider the cylindrical layer of concrete which envelops that steel.

Fig. 1.



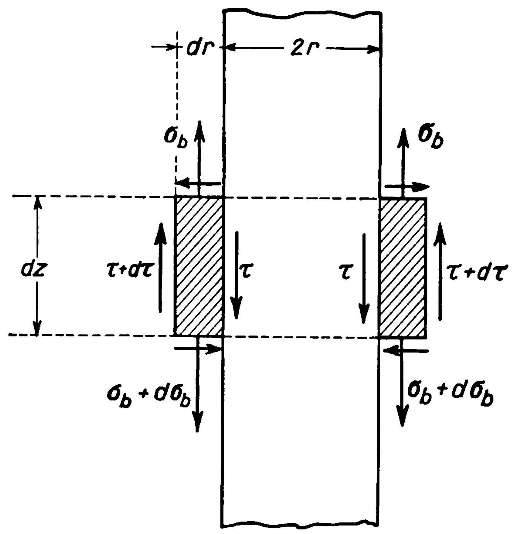


Fig. 2.

Assuming that dx is the minimum thickness of the layer, σ_b the normal sole tension to which it is subjected in relation to the first of the perpendicular cross sections considered, $\sigma_b + d\sigma_b$ the corresponding tension on the other perpendicular cross section, situated according to our hypothesis at a distance of d_z from the first one (Fig. 2).

The same considerations with regard to the equilibrium that we have just applied in the case of the steel when referring to this cylindrical concrete layer, lead us to a further formula which is the following:

$$d\sigma_b\,[\pi(r + dr)^2 - \pi r^2] = (\tau + d\tau) \cdot 2\,\pi\,(r + dr) \cdot dz - \tau \cdot 2\,\pi r \cdot dz.$$

Here we have naturally indicated by $\tau + d\tau$ the average sole coefficient of the tangential tension that the portions of concrete which surround the layer in question exercise on its external surface.

If we disregard the exceedingly reduced limits of a higher order than the second, this equation will take the following form:

$$d\sigma_b \cdot 2\,\pi\,r \cdot dr = \tau \cdot 2\,\pi\,dr \cdot dz + d\tau \cdot 2\,\pi\,r \cdot dz$$

or also:

$$\frac{d\sigma_b}{dz} = \frac{\tau}{r} + \frac{d\tau}{dr} \tag{2}$$

But if the cylindrical layer of concrete is to adhere absolutely to the concreted iron, it will be necessary to admit that on the surfaces which are in contact with each other, the deformations of the two materials are identical.

And thus, given E_f as the normal modulus of elasticity of the iron, and E_b that of the concrete, we should have:

$$\frac{\sigma_f}{E_f} = \frac{\sigma_b}{E_b} \quad \text{and} \quad \frac{d\sigma_f}{E_f} = \frac{d\sigma_b}{E_b}$$

In these circumstances, the ratio deduced from the coexistence of the two formulas of equilibrium shown above will be:

$$\frac{d\tau}{dr} = \frac{2E_b - E_f}{E_f} \cdot \frac{\tau}{r} \tag{3}$$

in which the coefficient:

$$\frac{2E_b - E_f}{E_f}$$

is always a negative one.

If, as happens in practice, it be admitted that

$$E_f = 10\,E_b$$

that coefficient will have the value of:

$$-\frac{4}{5}$$

In any case it is possible to state in a general way that the tangential tensions in concrete decrease fairly rapidly as soon as the distance from the surface of the steel is increased. The rate of decrease will be accelerated in proportion as the ratio $\frac{\tau}{r}$ between the maximum intensity attained by these tangential stresses on the above mentioned surface and its radius is greater.

But in connection with the first of the equilibrium equations given above, it must be remembered that the rapidity with which the σ_f vary (and accordingly the σ_b also) related to z, also depends on the coefficient of the ratio $\frac{\tau}{r}$.

In this way we are led to conclude that two conditions must occur if the transmission of the stresses from the steel to the concrete (or vice-versa) are to take place either longitudinally or transversally in a very limited zone. These conditions are:

(1) a high coefficient of τ which means satisfactory adhesion between the two materials;

(2) a low coefficient of r which means distribution of the metal section in a number of iron bars of small diameter.

The first of these is self-evident, while the second is a direct reminder of those experimental results to which allusion was made in the early part of this paper, and which enabled us to specify the double advantage to be gained by the use of iron parts of small diameter, an advantage which could be realized, according to circumstances, in that, given equal maximum tangential tension, the transmission of the stresses from steel to concrete (or vice-versa) can be effected in a zone with a maximum of limitation either longitudinal or transversal, or by the fact that where other circumstances are equal, this transmission of stresses will set up less marked tangential tensions.

Summary.

The Author describes the incompleteness of assumptions on which the calculation of reinforced concrete sections is based. He examines the transmission of tangential stresses and shows how this can be improved. The calculation offers a proof to the well known fact that a large number of thin reinforcing rods is preferable to a small number of heavy rods.

IIb 3

Practical Improvements in the Mechanical Treatment of Concrete [1].

Praktische Weiterentwicklung der Verfahren zur mechanischen Behandlung von Beton.

Progrès pratiques des méthodes de traitement mécanique des bétons.

E. Freyssinet,
Ingénieur-Conseil, Neuilly-s-Seine.

The considerations put forward in the first two parts of this paper are, as the Author will show presently, of some importance not only from a speculative point of view but still more from that of practical application on works. Starting from these theoretical grounds he has been led to the attainment of conditions which are unprecedented and which amount to a revolution in the combined usage of steel and concrete — conditions which render possible not only a large saving in the cost and weight of materials necessary to build a structure of specified dimensions and strength but which open up altogether new technical possibilities of the highest interest. These, it is clear, depend on the combination of steels of high strength and high elastic limit with grades of concrete likewise of high strength.

Difficulties attending the use of high-elastic limit steel.

A reduction in the cost of the steel required to build a structure of given dimensions and strength must depend on reducing the ratio $\frac{\text{price per unit weight}}{\text{ultimate stress}}$

[1] This paper by *Freyssinet* forms part of a work on "New aspects of the problems of reinforced concrete", the first two parts of which, briefly summarized below, appeared in the 4th volume of the Memoirs of the I.A.B.S.E.

In the first part *Freyssinet* developed a theory of strains in cement and concrete based on the principles of thermodynamics and on the hypotheses of physics. This was expressed in a series of theorems, valid for any substance of the kind referred to by him as a "pseudo-solid", which has the form of a network of very fine interstices which can be filled either by a liquid wetting their boundaries or by a gas.

In the second part, by introducing new hypotheses, the Author determines the general properties of cements and the mode of formation of the pore structure in cement pastes. From this he deduces some important practical consequences and in particular he explains why it is that the strength of a cement depends more on the mechanical and physical conditions of its use than on any other factor. He proceeds to show that with any cement a considerable degree of hardness can be obtained almost at once, simply through an improvement in the compactness.

which varies in the same way as the ratio $\frac{\text{price per unit weight}}{\text{elastic limit}}$. At a price slightly higher than that of the mild steels commonly used, the mills can supply steels having strengths close to 100 kg per sq. mm (63.3 tons per sq. in.) wherein the elastic limit can easily be above 80 kg per sq. mm (50.6 tons per sq. in.), and in response to large orders specially treated steels could be delivered with an elastic limit well in excess of 120 kg per sq. mm (75.6 tons per sq. in.).

Since the steels in use at present are confined almost exclusively to an elastic limit of 24 kg sq. mm (15.2 tons per sq. in.) it follows that the present ratio $\frac{\text{elastic limit}}{\text{price}}$ can be divided by a coefficient of about 3 which is capable of being increased in the future to more than 4. In large spans the coefficient would be higher still on account of the indirect advantage due to reduction in dead weight.

Unfortunately, if, in a reinforced concrete structure, the ordinary steel be replaced by steel of higher strength — for instance by steel having an elastic limit of 120 kg per sq. mm (75.6 tons per sq. in.) — very serious cracks appear as soon as the structure is so loaded as to cause stresses in the metal notably greater than the usual values, and the performance is little better than if the reinforcements were of ordinary mild steel of the same section. The reason for this lack of success is that in ordinary reinforced structures the strain in the concrete is practically equal to that of the steel carrying the whole of the tensile forces, and increases in proportion to the stress, Young's modulus being independent of the grade of steel. Consequently the limits of elongations that can be withstood by the concrete correspond approximately to those governed by the usual stresses in mild steel.

The use of high-strength steels working in tension therefore adds very little to the practical value of a structure, and being subject to certain disadvantages it is not a feature of current practice[2].

Difficulties attending the use of high-strength concrete.

The Author has explained how, through improvements in the density of concrete brought about by mechanical means, its compressive strength may be made nearly to equal that of the solid stone from the fragments of which the aggregate has been formed.

Since in quarries it is easy to obtain aggregates derived from rocks having crushing strengths of the order of 1000—1500 kg per sq. cm (6.3—9.5 tons per sq. in.) or even much more, it results that present values for the ratio $\frac{\text{price per cubic yard}}{\text{crushing strength}}$ can be reduced in approximately the same proportion as can the ratio $\frac{\text{price per unit weight}}{\text{elastic limit of steel}}$. It may be remarked incidentally that in

[2] The Author would, however, recall the conclusion emerging from his work on strains in concrete to the effect that the use of hard steel for compression bars is very effective and is strongly to be recommended.

concrete of this kind the ratio $\frac{\text{elastic limit}}{\text{density}}$ is much higher than in ordinary steels and *approximates to that of the best aeronautical metals*, especially if in this comparison the disadvantages of erection are considered which are non-existent for concrete.

In the conventional mode of association between steel and concrete, however, these qualities of price and specific lightness are ill exploited. Increasing the density of concrete leads to an increase in compressive strength but the tensile strength is much less affected by this and remains much more subject to the conditions under which concrete is prepared; and further still, the various possibilities of deformation are greatly reduced. Hence in the first place the value of the ratio $m = E_s / E_c$ is reduced, which raises the neutral axis and so increases the compressive stress in a member subject to bending, thus taking away much of the advantage sought.

Secondly, it is the case that tensile strength is at least as important as compressive strength, for the performance of a structure in respect of shear effects depends entirely on the former, as do many other destructive agencies which may come outside the scope of calculation but of which account must be taken in practice — a fact which engineers express by saying that in practice certain minimum thicknesses must be provided.

Thirdly, every structure of whatever kind, suffers what Mr. *Caquot* has called permanent adaptative deformations, which result in the stresses becoming uniform and their maxima being diminished: if however the concrete has an insufficient capacity for plastic strain this adaptation is not well accomplished.

For all these reasons the advantages that really accrue, in ordinary forms of structure, from the introduction of highly compacted concrete are much less than might be expected on the basis of compressive strength alone.

Theoretical conditions for fully utilising the qualities of high-strength concretes and steels. — The difinition of pre-established stressing, or pre-straining.

It follows that good usage of high-strength materials is governed in the case of the steel by keeping the elongations of the concrete within their usual practical limits, and in that of the concrete by keeping the total tensile strain well below the tensile breaking stress therein.

Theoretically this double condition can be satisfied by making use of the steel, not to carry such tensions as will produce in it elongations which the concrete cannot follow, but to cause permanent strains in the concrete in directions opposite to those resulting from the loads — that is, to produce compression in the tension zones and tension in the compression zones.

This result can be attained by subjecting the steel to tension before the concrete is poured: for this purpose the reinforcing bars are gripped near their ends by two temporary attachments and are pulled upon by means of jacks which take purchase on abutment members so as to produce a known amount of stress in the bars; after the concrete has hardened sufficiently to afford anchorage to the ends of the bars the jacks are removed and the tendency of the bars to contract imposes in the concrete a compressive force equal to the tension in the former. By another method, tension is created in bars anchored in concrete which has already hardened.

Thus there is developed in the steel-concrete system a double system of permanent strains similar but opposite in direction to that which arises in ordinary reinforced members as the result of shrinkage, the harmful effects of which are already known. A first consequence of pre-straining, then, is the disappearance of undesirable effects due to shrinkage. With this final aim in mind researches have been made in many countries and for a long time, to find the proper practical means, but all these trials failed completely, and were abandoned except that in hoop reinforcement for pipes a certain mode of application was found.

Causes of checks encountered in the first attempts-absolute necessity, for obtaining permanent counter-strains, of using very high-elastic limit steels and very dense concrete.

The ill-success prior to the present Author's researches was neglect of the laws of stress and strain in concrete on the part of those concerned. It was assumed that tensions of the order of a few kg per sq. mm in the steel should suffice to compensate shrinkage, while on the other hand the concrete used — being of the only kind which it was understood practically how to make — had a high water-cement ratio and was therefore very sensitive to all sources of strain, contracting, as the combined result of initial tension and of continual changes in hygrometric conditions, to a total extent greatly in excess of the elongation imposed on the steel. Hence the conclusion put forward, notably by Koenen in Germany, that permanent pre-strains are impossible of attainment.

It is now known that the total strains in concrete though greater than was previously supposed are nevertheless limited. The upshot of all the experiments is that the laws of contraction under load are represented by curves terminating in asymptotes along the time axis, the ordinates of which are governed principally by the water-cement ratio in the concrete at the moment of initial set. Consequently, if the steel is given an elongation such that the maximum contraction of the concrete is only a moderate fraction thereof, the greater part of the pre-strains so obtained are permanent and absolute reliance may be placed upon them even after very long periods of time.

In ordinary concrete the contractions may amount to 3/1000 or even more, depending on the conditions of shrinkage and load; this would imply a relief of tension in the steel which may exceed 60 kg per sq. mm (38 tons per sq. in.) and there is, therefore, no reason for surprise that tensions of 10 or 12 kg per sq. mm (6 or $7^1/_2$ tons per sq. in.) have given negative results.

By the use of very dense concrete the deformations therein, and consequently the drop of tension determined thereby, can be considerably reduced. The matter is one to be decided for each case in accordance with the particular data of the problem, the load on the concrete, its instantaneous and long-term mechanical properties, and the average hygrometric conditions; generally speaking the figure in question will fall between 10 and 30 kg per sq. mm (6 and 19 tons per sq. in.)

this upper limit being reached only where compression is of the order of 200 to 300 hectopiezes.

The Author is accustomed to use initial tensions so calculated as to produce permanent tensions of between $^1/_2$ and $^2/_3$ of the elastic limit of the metal which, in the applications hitherto made, has been of the order of 80—90 kg per sq. mm (50 to 57 tons per sq. in.).

Study of the changes in mechanical condition resulting from pre-strains in members subject to bending in a uniform direction.

a) *Stabilisation of strains in steel and concrete under tension due to varying loads.*

In the case of moments in a constant direction, there arises the need to subject the concrete which is under tension from the loads to permanent pre-compression of the same order as this tension and generally a little in excess. The concrete will then remain under such compression after the loads are applied.

The deformations occurring under varying load are governed by the inertia of the whole section including steel; in reinforced concrete they are governed by the inertia of the compression zone alone combined with the steel. That zone being only a fraction of the whole section it results that a first effect attributable to the counter-strains is a reduction in the deformation of the tension zones in a proportion which frequently reaches 5 to 1 by comparison with ordinary reinforced concrete.

Not only is the concrete relieved of all risk of cracking, but the contraction stresses suffered by the steel, which are proportional to a deformation equal to that in the concrete, vary much less than in reinforced concrete.

b) *Reduction of compressive strains in concrete compared with ordinary reinforced concrete.*

In reinforced concrete members the raising of the neutral axis above the centre of gravity, which results from the elastic elongation of the steel being greater than the elastic contraction of the concrete, brings about a very notable increase in the maximum compressive stresses by comparison with the values they would have in a homogeneous member of the same dimensions; the denser the concrete and the greater the strain in the steel the more pronounced is this effect.

In pre-strained members, however, the neutral axis of the concrete drops below the centre of gravity of the section whatever the density of the concrete, and the strains are in consequence much reduced. Hence if the concrete in an ordinary reinforced member is stressed equally with that in a pre-strained member the latter will carry much heavier loads than the former.

The advantage is considerable, and to show its order of magnitude the example will be taken of a reinforced concrete beam of rectangular section stressed at 50 kg per sq. cm (711 lbs. per sq. in.) compression in the concrete and 15 kg per sq. mm (9,5 tons per sq. in.) in the steel. Assuming $m = 10$ the neutral axis will be at $^1/_4$ of the distance between the extreme compression fibre and the reinforcement. By the introduction of pre-strain the neutral axis can be lowered as much as desired and may be brought close to the reinforcement: the compression zone will then extend over the whole of the beam and will be four times as large as in the previous case, and while it is true that the lever arm will thereby be reduced from $^{11}/_{12}$ to $^{2}/_{3}$ of the distance between the reinforcement and the extreme compression fibre yet the load carried with the same maximum stress of 50 kg per sq. cm will be increased in the proportion —

$$4 \times \frac{\frac{2}{3}}{\frac{11}{12}} = \frac{96}{33}$$

or nearly 3 times.

As a set-off against the improvement in the employment of the concrete, the force imposed on the reinforcement will be multiplied in the ratio $\frac{33}{24}$, but there can be little objection to this as the actual weight of the steel will still only be 33/84 of what it would be in an ordinary beam.

The effect of this reduction in stress will, of course, be all the greater if the logical step is taken of combining the application of pre-straining with that of high-density concrete. On these lines one may imagine slabs of a given thickness covering three times the usual spans, without, as here explained, the strains exceeding an acceptable magnitude. The practical importance of this conclusion will at once be appreciated as regards, for instance, the limiting spans of full-webbed plain beams or of mushroom slab floors.

c) *Shear.*

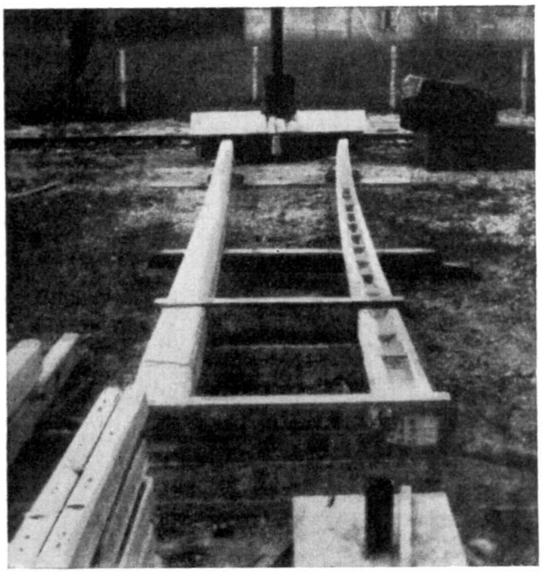
Pre-compression gives advantages at least equally great from the point of view of shear as from that of bending which has just been discussed.

To show these it is only necessary to plot Mohr's diagrams relating to simple shear and to shear accompanied by one or two tensile forces or one or two compressions. Such diagrams disclose how the stresses that accompany shear are exaggerated by the effect of shrinkage; how greatly on the contrary, the maximum tensions may be diminished by introducing a compressive force in one direction; and how the introduction of two strains at right angles can be made to bring about their total disappearance[3]. The most favourable system is that which contains two compressions equal to the amount of shear and in such a case the most dangerous strain is that corresponding to a compression equal in intensity to twice the shears stress. The limiting shear stress for use in design may then be equated to one half of the limiting compressive stress in the concrete, an assumption which will ensure excellent use being made of high-strength concrete and an enormous saving by comparison with the usual dimensions.

d) *Resistance to repeated loading.*

Ordinary reinforced concrete structures, like those in riveted steelwork, do not stand up well against alternating loads. Experience has shown that pre-strained members do so incomparably better.

[3] For the experimental confirmations of this fact see "Science et Industrie", January 1933.



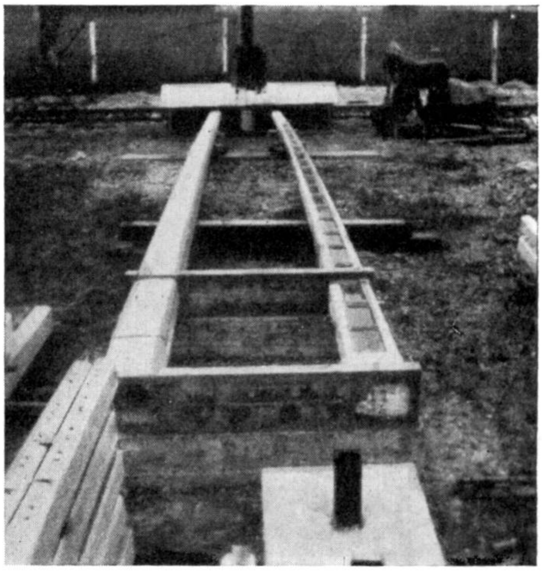


Fig. 1 and 2.

The posts A (left) an B (right) are subjected to forces which pulling them together an thrusting them apart.

Such alternating loads were applied to a pre-strained column A and an ordinary column B each 12 m (39 ft. $4^1/_2$ in.) high and embedded up to 2 m (6 ft. 6 in.) from their bases. (Fig. 1, 2, 3, 4.) Column A was 5 months old; it contained 50 kg (110 lbs.) of steel and weighed 750 kg (1654 lbs.). Column B was 18 months old; it contained 130 kg (287 lbs.) of steel and weighed 980 kg (2160 lbs.). The breaking load as measured on columns identical with these was about 900 kg (1984 lbs.) in each case, and the load actually applied to

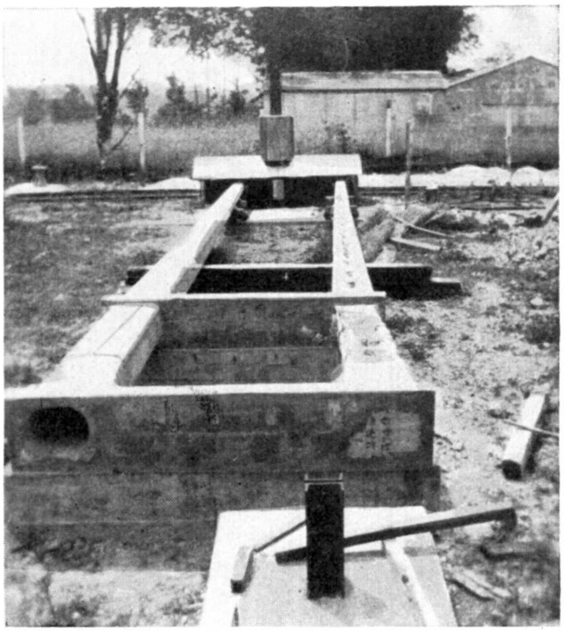


Fig. 3.

The posts A and B at the conclusion of tests (Posts B completely fractured).

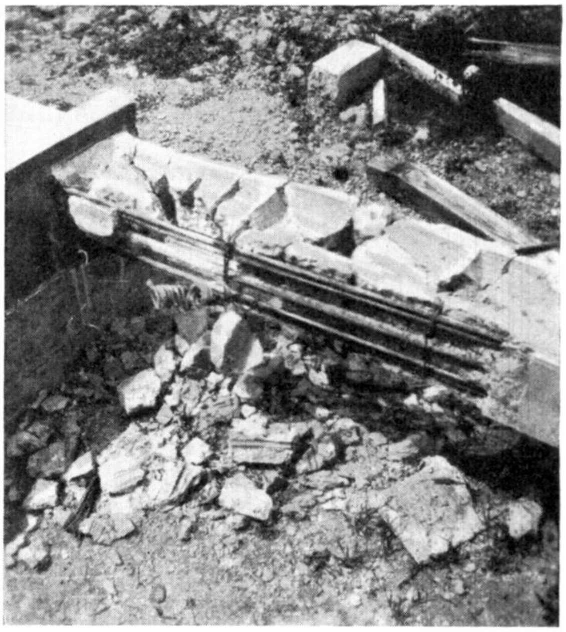


Fig. 4.

Close-up of post B after testing.

their ends was varied between — 450 and + 450 kgs (± 992 lbs.) about 8 times a minute.

After a few hundred alternations Column B was heavily cracked and after a few thousands it was completely broken. Column A, on the other hand, resisted 500 000 alternations without measurable damage.

General conditions for the practical use of pre-straining: — The necessity for applying the stresses at very low cost.

It has been shown theoretically possible to subject reinforced concrete members from the time of their construction to such mechanical conditions as will allow of fully realising the qualities possessed by high-strength steel and concrete. It has further been explained that in this way various advantages may be gained such as great reductions of the maximum compressions in members subject to bending and of the maximum tensions in members subject to shear; also a greatly improved resistance to repeated loading.

To convert these possibilities to actualities two problems must be solved: firstly that of stressing the steel, secondly the practical manufacture of high-strength concrete at a cost low enough to maintain as large as possible a share of the savings in materials.

The cost of ordinary steel reinforcement having an elastic limit of 24 kg per sq. mm (15 tons per sq. in.) may be estimated at 1 franc per kg including preparation. Reinforcement weighing 3,5 kg per metre (2,35 lbs. per ft.) will, therefore, cost 3,50 francs per metre, and it can be replaced with the same factor of safety by reinforcement of steel having an elastic limit of 84 kg per sq. mm (53,2 tons per sq. in.), weighing only 1 kg per metre (0,67 lbs. per ft.) and costing 0,90 franc as supplied; this, however, must be tensioned up to about 8000 kg (7,9 tons). The saving, therefore, is 2,60 francs per metre less the cost of the following operations:

1) Cutting of bars and preparation of their permanent anchorages in the concrete.

2) Arrangement of temporary anchorages at two points within the forms near the ends of the bars to withstand a pull of 8 tons.

3) Production of this pull of 8 tons between the anchoring points, and maintenance of it while the concrete is being poured and is hardening.

4) Dismantling of temporary anchorages buried in the concrete within the forms and transfer of the tension carried by them to the permanent anchorages against the concrete; making good the holes left in the concrete after dismantling.

In order to result in a saving the cost of these various operations must be less than 2,60 francs multiplied by the length of the bar in metres, and it is further to be borne in mind that the removal of shuttering will nearly always have to be delayed until after the concrete has been placed in compression, which implies and advanced stage of hardening; the costs attributable to an increased period of immobilisation of the forms must, therefore, also be taken into account.

Except where the reinforcing bars are very long the margin calculated on these lines is very small, and the whole practical problem of using high-strength steels may be said, therefore, to hinge upon a reduction in the cost of the means of anchorage and of tensioning the bars.

As a rule the cost of tensioning increases less rapidly than the amount of tension produced; it is scarcely at all affected by the length of the bars; on the contrary the margin for economy rises in proportion to the cube of the linear dimensions. It follows that, contrary to what one might expect at first sight, the greater the absolute dimensions of the members and the heavier the forces to be developed the easier to solve are the practical problems involved in tensioning the steel bars. As regards small members wherein the forces are

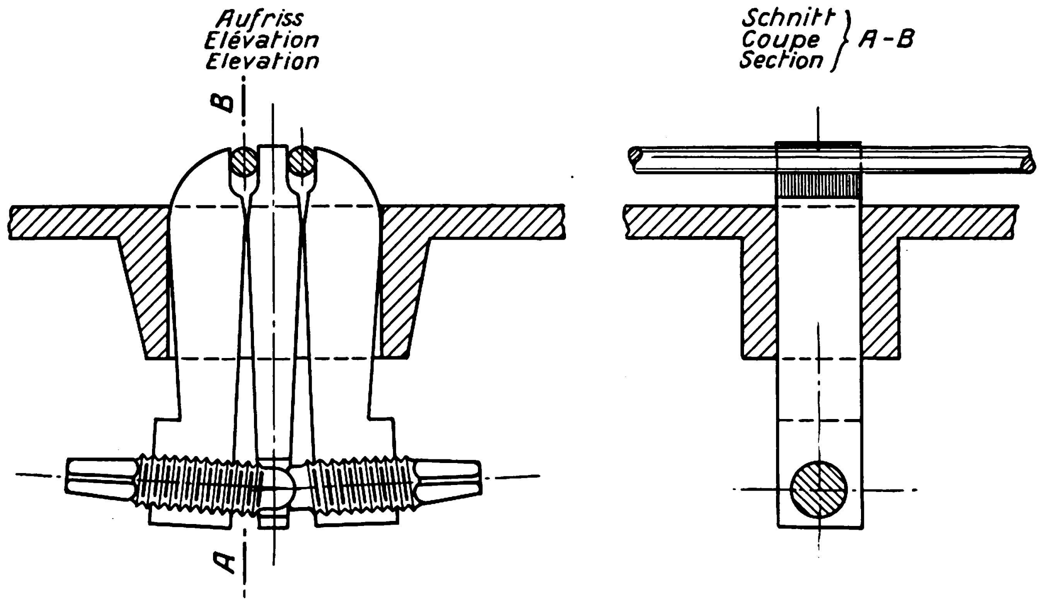


Fig. 5,

Jaws for clamping bars.

small there is no satisfactory solution except in the case of repetition work on very long series.

As the anchorages must offer the same resistance as the bars themselves so as not to form a link weaker than the latter, most of the usual types of anchorage are ruled out including those which depend on a screw and nut. The Authors has, however, successfully made use of fixtures in which the steel bars are wedged between jaws or corners or are secured by loops formed by electric flash welding (Fig. 5).

Frequently there is a lack of space between the reinforcing bars, so that the size of the anchoring arrangements has to be reduced to a minimum: this implies the use of metals of the highest possible quality so treated as to produce the maximum improvement in their tensile strength and resistance to wear. Difficulties are avoided by enclosing the bars in pre-cast concrete pieces held in place against the shuttering either by simple adhesion or by means of suitably arranged grooves, or by other methods depending on the special conditions of each case.

Relationship between the problems of tensioning the steel bars and the rapid hardening of the concrete.

The necessity for very low costs in tensioning the bars has led to a search for improved gear minimising the labour involved. But such gear has to withstand very large forces; this makes it expensive and it is immobilised throughout the period of hardening of the concrete. Hence the great importance of shortening this period; it may be said, indeed, that the possibility of practical application for pre-straining is linked with the development of processes to secure rapid hardening of the concrete.

The Author has already described the means whereby he has been able to combine very intense hardening with great speed. These means consist of treating the concrete by vibration and compression, and in the case of Portland and slag cements by heating to about 100^0 C (see below). With good Portland cements the time required to obtain sufficient strength to withstand pre-straining is about $1^1/_2$ hours from the filling of the forms. In the case of slag cements very good hardening is obtained in 2 to 5 hours according to the cement and the heating conditions.

The heating is accomplished very easily by means of steam, suitable arrangements for the purpose being made in the forms. The temperature of the concrete is sometimes considerably higher that that of the steam in consequence of the heat of reaction, and for the same reason the steam consumption is low, only some 10.000—20.000 calories having to be added to raise a cubic metre from 10^0 to 100^0 C; in practice the cost of heating the concrete is no more than a few francs per cubic metre.

One result of instantaneous hardening of the concrete is that members can be built up from elements successively deposited along continuous reinforcements which are tensioned after the pouring and hardening of certain parts only of the members, thus requiring only a limited amount of gear for the purpose which is easily managed and not expensive. It will be shown that this procedure is capable of very wide application.

Supply of high-elastic limit steel.

Each application must be preceded by arrangements for the supply of steel bars of suitable quality at a price and in a form which will make their use feasible in practice. Such steel bars must have a high and uniform limit of elasticity, must not be brittle and must be perfectly straight. This last condition is very important because the straightening process usually applied on the job ist impossible in the case of high-elastic limit reinforcements which act like springs.

In France, high-elastic limit steel bars are not commercially available at a price of the same order as ordinary concrete reinforcing bars. Drawn rods are too expensive and do not give good adhesion to concrete. The commercial metals that have properties closest to those required in the applications here contemplated are machine-made rods giving a breaking stress, in their condition as supplied, of some 100 kg per sq. mm (63 tons per sq. in.) but an elastic limit, in that condition, very variable and sometimes scarcely higher than that of mild steel. These are supplied in coils often of irregular shape and weighing 50—150 kg (110—330 lbs.), in diameters up to 16 mm ($^5/_8$ in.), at a price which, mainly because of the limited demand for metal of this kind, is at present slightly higher than that of ordinary round reinforcing bars.

These coils require, then, to be converted into straight wires of high elastic limit without appreciably adding to their cost. The Author has produced machines capable of effecting this conversion at a cost of some centimes per kilogramme (Fig. 6). To avoid any loss through short lengths the end of each coil is flash-welded to the beginning of the next, the weld being subsequently annealed

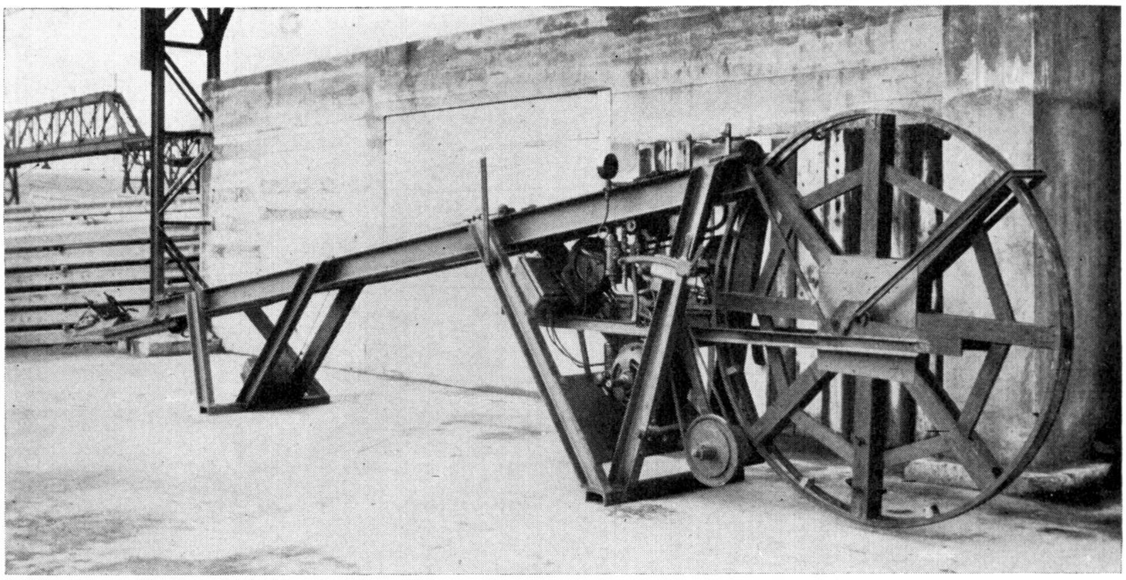


Fig. 6.

Device for increasing the yield point of steels.

by the welding machine itself — operations which take up only a few seconds and which produce a weld having exactly the same strength as the wire. The wire is next drawn into the machine where it is first roughly straightened between rollers in two planes at right angles and is then stretched between pincers closed by hydraulic jacks, n metres apart: one of these pincers is fixed and the other mounted on a trolley moved by a screw so as to pull on the wire; immediately the n metres of wire have been tensioned to the desired amount a frictionless valve operates to release the engagement with the screw and return the trolley to its original position at a speed determined by an adjustable hydraulic brake. The accuracy obtained in tensile stressing by this means is of the order of 1 %.

A fraction $\frac{n}{p}$ of the wire is then drawn out beyond the machine, where it can either be kept straight or be coiled to a radius such that only an elastic strain is set up. The operation then automatically begins over again. The welds, too, pass through the machine and undergo the same stretching treatment. A wire of unlimited length is thus obtained which has an elastic limit approximately equal to the tensile stress exerted and in which every point, including the welds, has been testet to this intensity of stress P times over.

Elastic limits of between 80 and 90 kg per sq. mm (50 and 57 tons per sq. in) are easily obtained, which is $3^1/_2$ times as high as in ordinary steel bars. In the course of the operation the wire is lengthened by about 5 %.

Given a sufficient demand for hard wire the mills would no doubt be able to make direct deliveries of metals with chemical compositions specially designed to favour very high strengths, ready stretched, tempered, annealed and stretched over again, for which the ratio $\frac{\text{price}}{\text{elastic limit}}$ would be very notably less than can be obtained at present; such wires with a diameter of the order of 16 mm ($^5/_8$ in.) would be equivalent in strength to ordinary steel bars of about 35 mm ($1^3/_8$ in.); they could be easily transported in coils of large diameter wound in such a way as to produce in them only elastic strain so that when unrolled they would be practically straight.

The use of metals of this kind involves special problems of detail — since cutting, the formation of hooks, etc. cannot be done in the same way as on ordinary bars — but none of these problems involves any real difficulty such as might increase the cost.

Applications.

The applications achieved by the Author fall under two distinct heads:

1) pieces or structures moulded all at once, such as electric transmission line poles, sleepers, beams of limited dimensions, pipes formed in the factory in separate lengths;

2) the much more important case of cylindrical or quasicylindrical constructions (using this term in the most general sense) carried out in successive lengths by means of a mould which travels continuously along the work in hand.

The Author has given a summary account of the manufacture of electric poles by his methods in Science et Industrie for January 1933, which he will not repeat here. Today the development of nearly automatic plant for such work has attained almost absolute perfection, and the Author has been able to manufacture railway sleepers in a similar way. These machines turn out concrete pieces with strengths exceeding 1000 kg per sq. cm (14,220 lbs per sq. in.) after being subjected to an initial compression of 100—300 kg per sq. cm (1422—4267 lbs per sq. in.) according to circumstances, and the surface is made perfectly smooth and compact.

An application falling under the second of the above headings will now be described in some detail. This was carried out in the course of the renewals to the sub-structure of the Transatlantic Station at Havre, where it derives importance equally from the technical difficulties encountered and overcome and from the great value of the work it was possible to protect by these means. The station in question is about 600 m (1968 ft.) long by about 55 m (180 ft.) wide, and over the greater part of its area it includes two floors each loaded to 2500 kg per sq. m (512 lbs per sq. ft.) in addition to a terrace. It has been founded on piles cast *in situ* reaching down to level 0.00, the level of the quays being + 9,50 m (+ 31.17 ft.); these piles pass through a recent filling of dredged material overlaying a shingle beach of no great thickness which in turn rests upon mud down to level — 20 m (— 66 ft.) where a very firm bed of gravel exists (Fig. 7). One of the long faces of the building is solid with a quay founded on the gravel, and in front of this the mud has been dredged down to level — 12 m (— 39 ft.) — that is, 22 m (72 ft.) below the edge of the quay — in order to form a tidal basin. A loading of 6000 kg per sq. m (12,289 lbs. per sq. ft.) is specified over the ground floor.

After the main part of the work had been completed it was found that the ground into which the foundation piles had been driven was sinking together

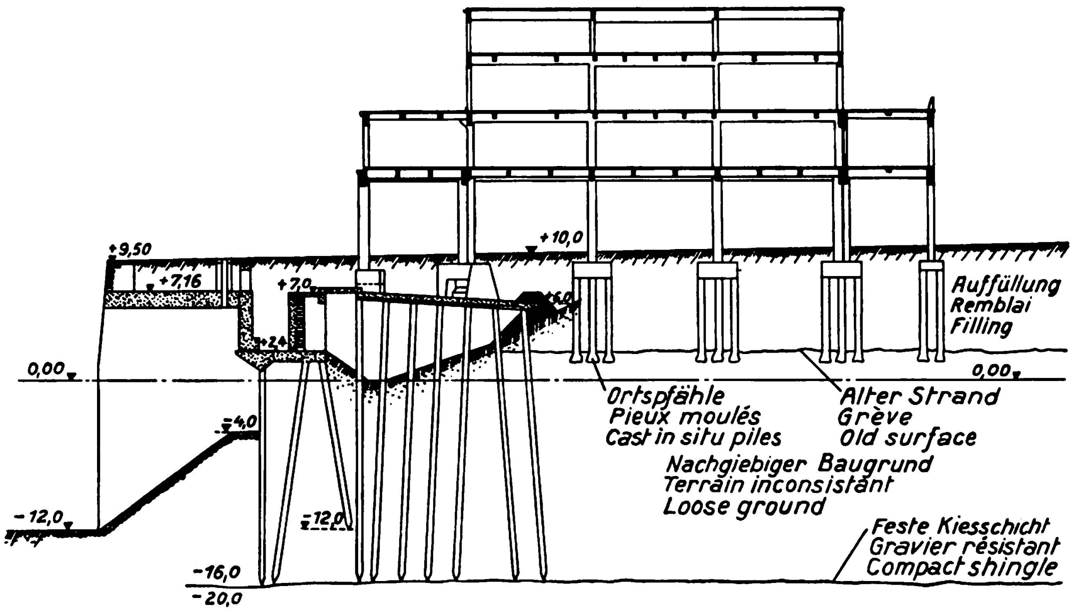


Fig. 7.

Cross section through Dock yard Station in Le Hâvre.

with the building in a single mass, its time law being nearly linear and the rate being of the order of one centimetre per month. It was essential that these movements should be arrested within a very short time. Only one method of doing so could be contemplated: the weight of the building, that of its old foundations and that corresponding to possible live loads together with a proportion of the weight of the filling sufficient to restore equilibrium in the mud where this had been disturbed by constructing the quay, by filling and dreadging, and by imposing the weight of the building must all be picked up and transferred unto the solid beds encountered at about — 20 m (— 66 ft.). This implied the placing of piles 30 m (98 ft.) long.

The underside of the ground floor, however, is about 5 m (16 ft.) above the ground, and water is met at a small depth; pile driving would not be possible as this would allow the mud to flow, thereby impairing the stability of the building and perhaps of the quay: hence the only admissible procedure was pressure piling by means of jacks.

Other possible dangers lay in so affecting the equilibrium of the subsoil as to prejudice the system as a whole: the mud might become liquid as the result of its repeated handling, and pressures might develop owing to the reduction in volume caused by placing the piles.

It was necessary, therefore, to have some means of controlling the pressures due to the placing of the piles. Further it was known that obstacles would be encountered which would have to be broken up, and beds of gravel which would have to be dredged. The heavy loading and the space occupied by the old foundations were such that most of the new piles must unavoidably receive loads of the order of 200 tonnes each or often more. The aggregate of the forces to be picked up at level — 20 m (— 66 ft.) under these conditions was in excess of 150,000 tonnes, equivalent to the driving of no less than 60 km (37 miles) length of large ordinary reinforced concrete piles carrying 75 tonnes each. Very powerful weapons of attack were, therefore, demanded.

Principle of the solution now in hand.

Trough the combined application of the pre-straining method and of rapid hardening of concrete the Author was able to improvise a solution to this problem, which, within about eight months of the decision to undertake the

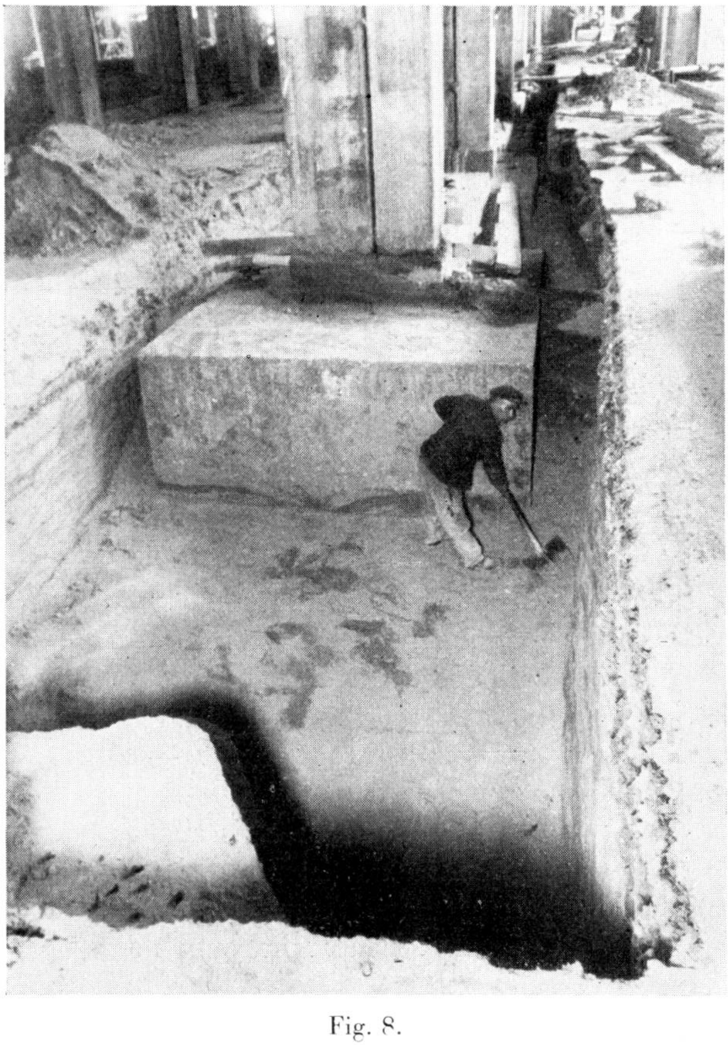


Fig. 8.

Present state of foundation slabs.

work and four months of its effective beginning, has stopped the settlement of those parts of the structure which were sinking at the most rapid rate — an effect which if continued must soon have exposed the building to a danger which has now been entirely removed.

a) *Connecting beams between existing-foundations.*

The columns of the building were originally carried on reinforced concrete footings up to 4.40 × 3.40 × 1.40 m (14.4 × 11.2 × 4.6 ft.) in size (Fig. 8 and 9). A beginning was made by forming these footings into large continuous

Fig. 9.

Ties between foundation during concreting Foreground showing form work lining pits for piling and for fixing ties.

girders to carry the foundation loads unto the new piles and furnish supports against which to jack down the pressure piles. This was accomplished by the following means. Masses of concrete, reinforced only against secondary stresses, were formed between the footings. The systems so constituted were placed in a general state of compression by means of steel tie bars wherein the elastic limit had previously been brought up to 80 kg per sq. mm (50.6 tons per sq. in.) and which were now tensioned to 50—60 kg per sq. mm (31—38 tons per sq. in.) between anchorages set in concrete headblocks at each end, one such anchorage fixed and the other actuated by jacks. Then, after packing tight so as to render the stress permanent, the jacks were removed (Fig. 10 and 11).

By these means a construction is produced which will resist considerable bending moments, shears and torsions, and this is done without either disturbing the sub-structure or making any important alterations to the existing footings, the concrete and reinforcement of these being turned to account.

Cylindrical cavities are formed running horizontally through the concrete cast in this way (Fig. 12); across each of these a pile is continuously manu-

Fig. 10.
Reinforcement for ties,

factured and is pressed into the ground at the same rate as it is made, this being accomplished by the operation of jacks which are bolted to the beams and which act upon collars capable of being rigidly attached to the piles as required.

b) *Description of the piles.*

The piles in question are hollow cylinders of 0.60 m (1 ft. $11^5/_8$ in.) external and 0.37 m (1 ft. $2^1/_2$ in.) internal diameter, the effective section being 1750 sq. cm (271 sq. in.). They are reinforced longitudinally by eight hard steel wires of 8 mm ($^5/_{16}$ in.) diameter and transversely by hooping of similar wire 6 mm ($^{15}/_{64}$ in.) in diameter. The total weight of reinforcement is 10 kg per metre run (6.7 lbs. per ft.) of the pile. Despite this low weight of steel the piles withstand a pressure of over 300 tonnes combined with a bending moment of 50 tonne-metres (161 ft.-tons), which constitutes a record.

c) *Method of forming the piles.*

Suppose a pile to be completed as far as the element N. The longitudinal reinforcements are delivered in unlimited lengths wound in coils to a diameter so large as to cause only elastic deformations, and normally they are continuous throughout the length of a pile (Fig. 13). The external mould consists of

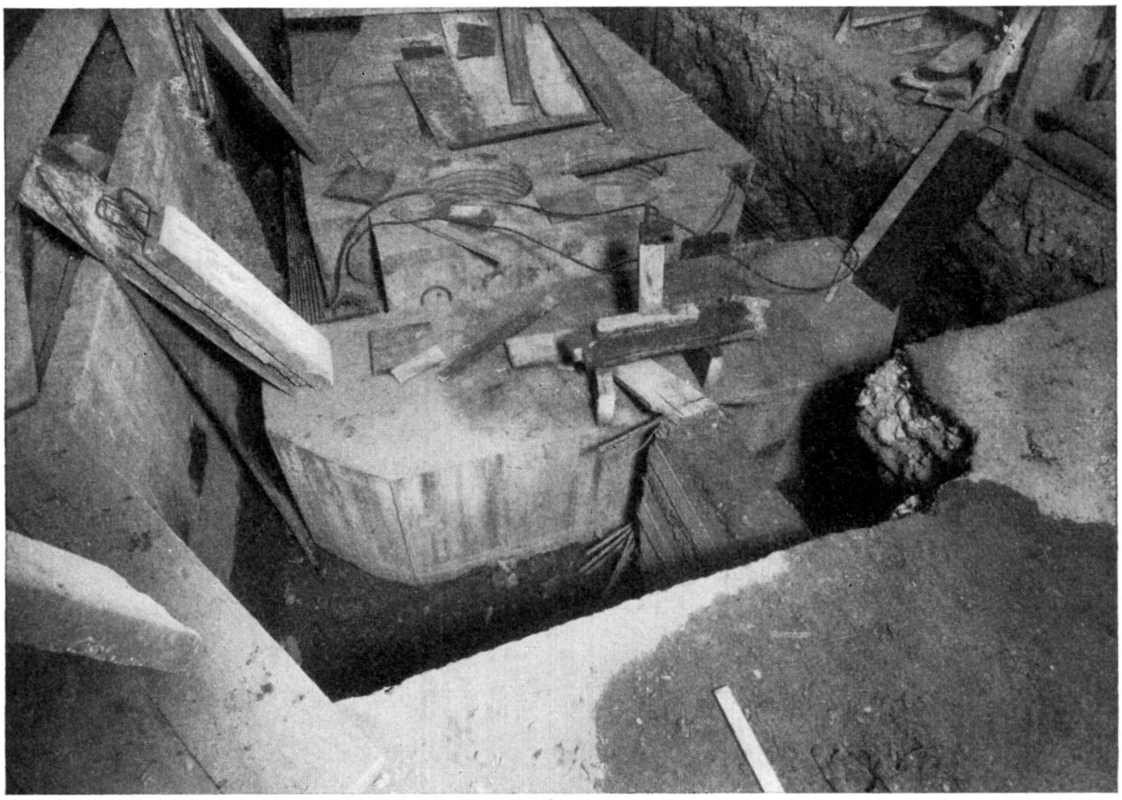


Fig. 11.

Movable head of tie during prestressing and wedging.

a set of 5 to 8 cylindrical sleeves each 0.40 m (1 ft. $3^3/_4$ in.) high and each split into two semi-circles with machined ends which can be pressed together by means of screws. (For the upper portion of the pile the sleeves have horizontal grooves inside them so as to produce horizontal ridges on the pile.) The internal mould consists of a steel tube covered by an envelope of rubber reinforced with cotton, and the bottom end of this tube is extended in a smaller diameter, also covered by a rubber pocket, so as to form a watertight cylinder (or obturator) normally of the same diameter as the steel tube but arranged to swell under hydraulic pressure. The space between the inner and outer moulds is closed at the top by an annular plate, through which are drilled 8 holes to pass the longitudinal reinforcements and 4 passages for filling.

When the lift N has been completed the sleeve-connecting presses are released in turn as the pile sinks, with the exception of the uppermost set. The mandril is raised through the height of one lift and the longitudinal reinforcements are bound to the spiral of the transverse reinforcement; then the sleeves forming the external mould are placed in their new positions; the longitudinal reinforcements above where they pass through the closing ring are seized between pairs of jaws mounted on supports which can be lifted by screws so as temporarily to stretch the reinforcements on which the rigidity of the pile depends; then the pocket on the internal mandril is filled under pressure so as to prevent any leakage of concrete taking place between the sleeve of the mandril

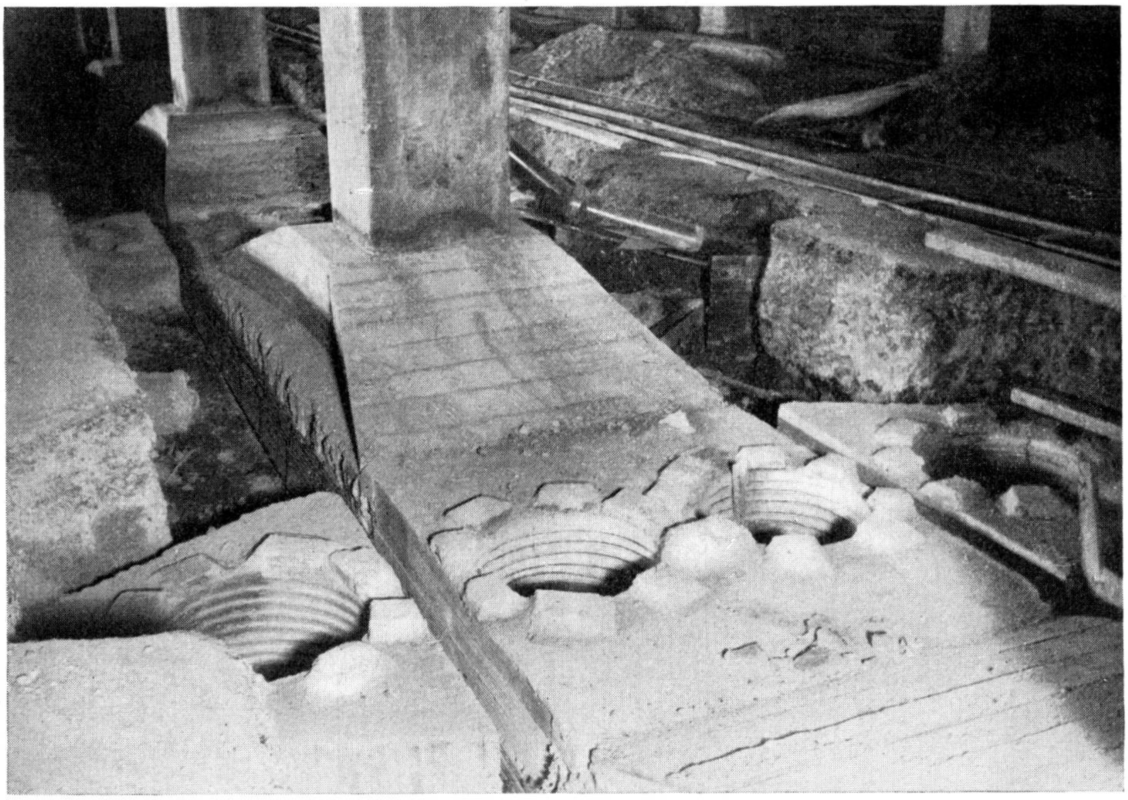


Fig. 12.

Group of 4 pile holes. The base for compressing the foundation will be placed in the slots seen left and right of the concrete foundation.

and the inner surface of lift N and so as to make good the joint between the steel tube and the sleeve.

These preparations being completed the mould is filled with concrete containing 450 kg of marine Portland cement per cu. m (28 lbs. per cu. ft.). The concrete is strongly vibrated by means of eccentric-mass electrical vibrators attached to the outer shell. Part of the excess water escapes through the joints between the sleeves and part rises to the upper surface which thereby tends to become softened, but the concrete is rendered homogeneous by strong pressure applied by screw plungers in tubes passing through the filling holes, vibration being maintained meanwhile. In this way excess water is expelled from the top portion of the pile. When this has been accomplished the filling holes are closed, vibration is discontinued, and water under a pressure of 20 kg per sq. cm (285 lbs. per sq. in.) is admitted between the mandril and its sleeve; immediately after the vibration the concrete acts as a fluid it transmits this pressure hydraulically to the plate at the top, lifting the latter and so stretching the steel bars to an extent approaching their elastic limit. This pressure is maintained for 20 minutes.

All the joints between the rings open and leak freely, and the concrete becomes very dry. The mould is surrounded by a heat retaining envelope into which steam at atmospheric pressure is admitted; in this way the temperature of the concrete is rapidly raised to over 100^0 C and a degree of hardness comparable to that of very good ordinary concrete after several months is reached at the end of a few hours even though the special cement for setting under sea water here used is very slow and normally gives rather poor final strengths (Fig. 14, 15, 16).

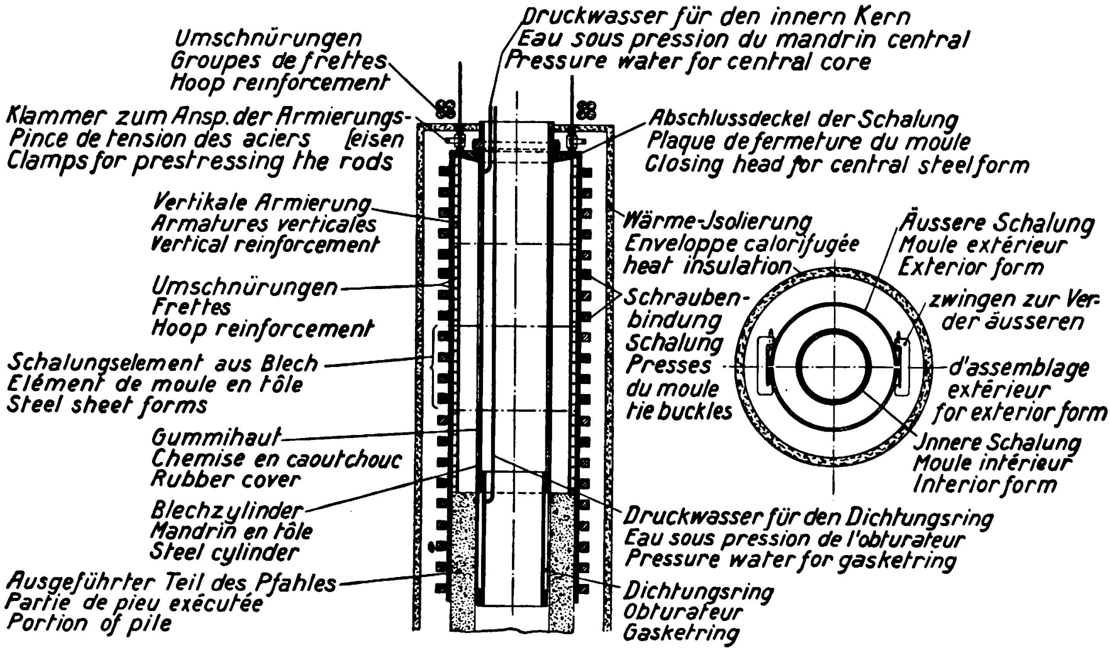


Fig. 13.
Formwork design for piles.

d) *Sinking the piles.*

Sinking follows immediately. Theoretically this calls for compressive forces up to 320 tonnes, to which must be added 20 kg per sq. cm (285 lbs. per sq. in.) of pre-strain, making a total of some 200 kg per sq. cm (2850 lbs. per sq. in.). It is shown by laboratory tests that strengths of the order of 300 kg per sq. cm (4267 lbs. per sq. in.) are reached after three hours heating in steam at 100^0 C; provision must be made, however, for bending moments — the existence of which is confirmed by experience — and these may reach a considerable magnitude, especially in the most recently executed zones. Now a moment of 50 tonne-metres (361 650 ft.-lbs.) will increase the stress to 500 kg per sq. cm (7112 lbs. per sq. in.), and the breakage of a pile in the ground would amount to a serious accident: the sinking is so conducted, therefore, as not to impose very heavy stresses on concrete less than about eight hours old. Practically no breakages of the piles have in fact occured except for a small number of mishaps which took place mainly during the period of initiation of the men concerned.

The equipment used for sinking consists of a collar formed of a water-tight tube concentric with the pile, inside which is arranged a rubber shield reinforced with cotton so as to give great strength and form a water-tight annular space between it and the tube. Between the rubber and the pile are placed staves

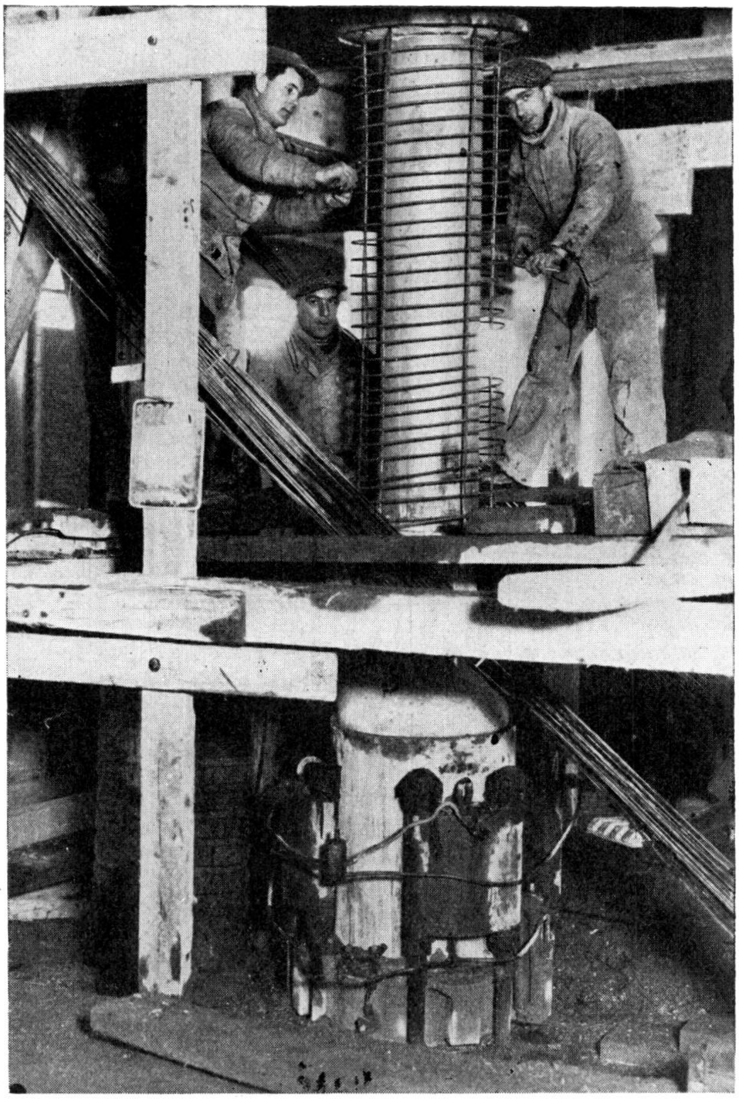


Fig. 14.
Reinforcement of piles (the lower position shows the screw winches).

parallel to the pile and almost touching one another, the parts of the staves in contact with the pile being of steel. When water under a pressure of 30 kg per sq. cm (427 lbs. per sq. in.) is admitted into the closed space the staves are pressed against the pile with a total force of 100 tonnes, and since the coefficient of friction between concrete and steel is over 0,40 a longitudinal adhesion sufficient to transmit at least 400 tonnes is produced between the staves and the pile. On removing the pressure all connection between the pile and the staves is destroyed (Fig. 17).

The staves are in contact with steel collars of which the upper one is very strong and serves to transmit the sinking pressure while the lower one serves for removing the staves and, if need be, for withdrawing the pile. These collars are welded to the tube and act in one piece with it. The upper collar has four welded lugs which receive the thrust of a corresponding number of pistons actuated by hydraulic jacks (Fig. 18), attached to the beam through which the pile is being sunk by means of eight hard steel bolts screwed into concrete, the

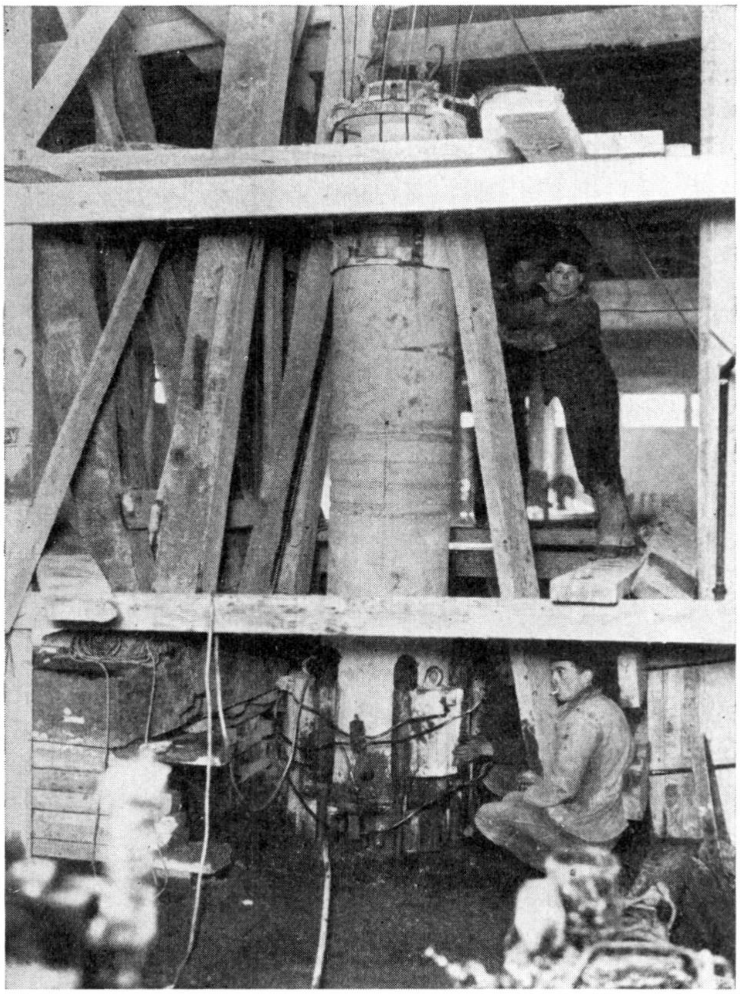


Fig. 15.

Piles after removing of formwork (lower position showing screw winches).

threads for this purpose being moulded in the actual concrete of the beam. Each of these bolts carries a load of 40 000 kg, but it has been confirmed by experiment that twice this amount could be withstood without giving rise to any trouble in the concrete which acts as a hold-fast.

The sinking operation is as follows:

1) The staves are pressed against the pile by admitting water at 30 kg per sq. cm (427 lbs. per sq. in.) into the sinking collar.

2) The pile is loaded by the simultaneous action of the four sinking jacks. Under a load which may reach a maximum of 320 tonnes no difficulty arises

in sinking the pile unless some obstacle is encountered, in which case this can be broken up or dredged out through the central cavity.

3) When the jacks have reached the limit of their travel the pressure in the sinking collar is released and the latter is raised by means of two special small jacks.

The cycle is then repeated.

When sinking is completed the pile is subjected to numerous alternations of 300 tonnes load on and off, and a check is then made as to whether any

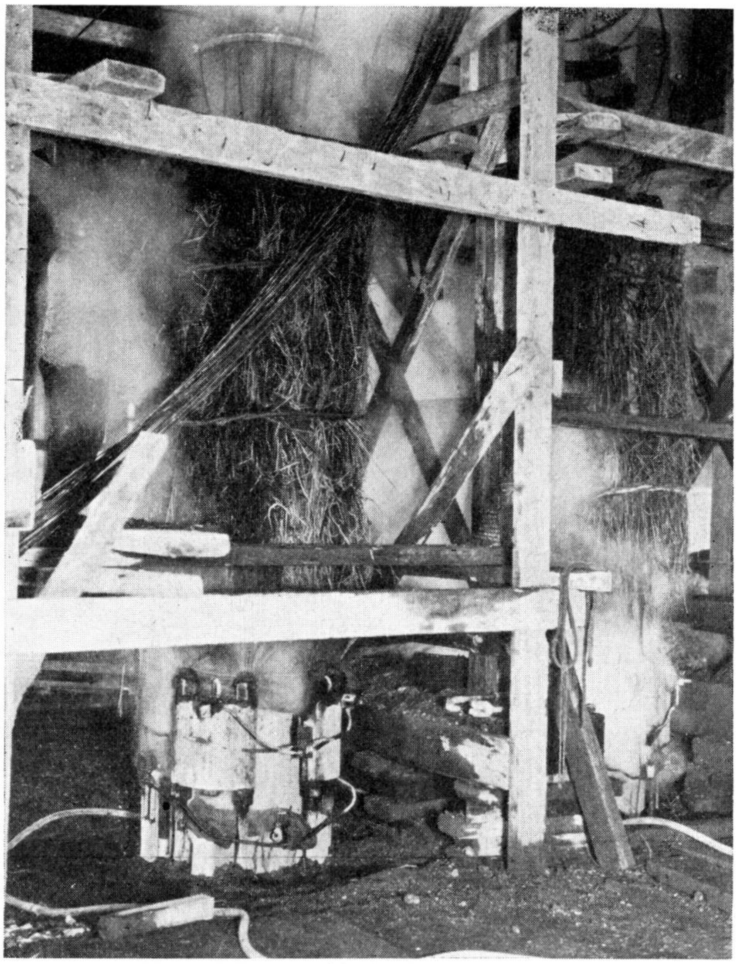


Fig. 16.
Two piles during heating.

appreciable sinkage occurs when the load of 300 tonnes is maintained for several hours on end. Generally speaking a definite refusal is soon obtained but in the case of certain of the piles these settling-down operations may take up a considerable time, perhaps several days, the pile continuing noticeably to go down little by little and its diameter changing under the effect of the alternating loads. Once the pile has settled down, concrete is poured into the annular space between the grooved pile and the grooved wall of the sinking space, and this concrete is allowed to harden while the pile remains under load. For the purpose of this last operation the sinking collar is replaced by an end plate.

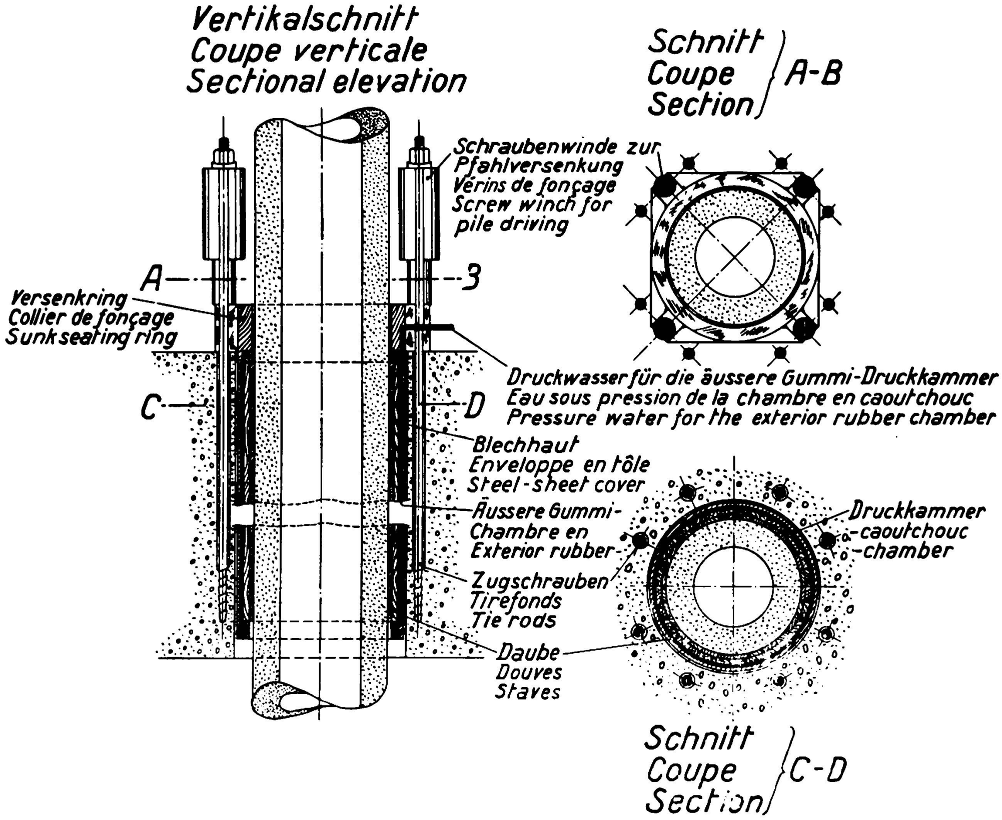


Fig. 17.

Sinking device for piles.

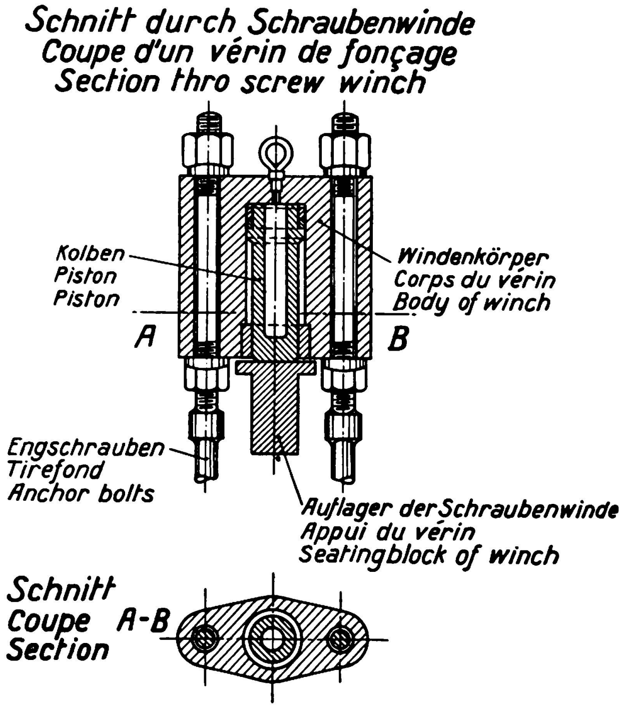


Fig. 18.

Detail of screw winch.

The piles formed in this way are perfectly true to dimension within and without, and generally they keep very straight. Sometimes, however, they are deflected by obstacles which bend them elastically and imply considerable bending stresses. As a rule these bends disappear if the pile is left alone, owing to the slow deformation of the ground actuated by the elastic forces.

Unless special obstacles are encountered a pile 30 m ($98^1/_2$ ft.) long may require about four days for its manufacture and sinkage. The work as a whole is proceeding under satisfactory conditions as to speed and cost.

Various applications of the Havre system of piling.

The methods described above are obviously capable of variations which would allow them to be applied to the making of any kind of pieces of practically cylindrical or prismatic shape, such as piles or any other members intended to be driven, screwed or forced in any possible way; columns or posts above ground; tunnel linings, inverts, walls, sheathings, floors, beams, arches, pipes, reinforced concrete roads, etc.

In almost every case some handy form of equipment can be designed which will be semi-automatic in its operation and will entail only a very limited expenditure on labour and on depreciation of moulds. Generally speaking the period of re-use of such equipment can be reduced to a few hours, or sometimes even to a few tens of minutes, allowing very high speeds of working with relatively simple plant. The economic advantages obtainable in these various applications can readily be inferred from what is said above.

One case of special interest is that of the pipe. The author has set up plant whereby pipes may be made automatically either in the factory or on the site, in a trench or underground, without joints, to any desired diameter, with both the transverse and the longitudinal steel bars all tensioned to their limit of elasticity. In this way a high degree of imperviousness is obtained under pressures which are limited only by stresses in the reinforcement of the order of 80 kg per sq. mm (50 tons per sq. in.). In the laboratory, staunch pipes have been obtained under pressures of 250 kg per sq. cm (3556 lbs. per sq. in.).

The resistance to bending and to possible shear is ten times greater than in the best made ordinary pipes of the same thickness. On account of the extreme degree of compactness the chemical resistance is remarkable, and the rigorous smoothness of the internal surface ensures a maximum discharge.

Another application which offers considerable scope for development is that of the mushroom-type floor. Under present conditions the span of such floors scarcely exceeds 30 to 40 times their thickness, whereas by the Author's methods this ratio can easily be doubled without adding either to the cost per unit of area or to the magnitude of the deflections. Mushroom floors are still looked upon as somewhat exceptional but by these means their use may become almost universal, especially in dwelling houses where it would considerably simplify the construction.

Long span girders in reinforced concrete.

In a number of earlier publications the author has indicated how reinforced concrete applied in the form of arches will allow the attainment of great spans such as can be exceeded only by suspension bridges. The methods that have just been described clearly enable the theoretical limits of span for such arches to be increased still further, while moreover the possibility of almost instantaneous hardening means that entirely new methods of construction are made available so as to extend the field of use for concrete into regions previously regarded as the preserve of steelwork alone. This subject, however, is so wide that its treatment must be relegated to a separate work.

The Author will confine himself here to the matter of straight girders. Until now it has not been possible to contemplate the economical use of reinforced concrete in the form of straight girders of large span, particularly in the case of relatively shallow solid-webbed girders. There are three reasons for this: the unsatisfactory use made of the concrete under compression; the impossibility of utilising any high degree of strain in the steel and the consequent necessity for a large amount of steel surrounded by a heavy concrete casing; and above all the very poor use that could be made of concrete in the members transmitting shear forces.

At all three of these points the Author's methods are capable of supplying a considerable improvement. He has found from the study of particular cases that by making use of these new forms of technique the limiting spans that can economically be obtained with reinforced concrete girders may be multiplied by a coefficient of the order of five to ten. Solid webbed girders of 100 m (328 ft.) span become feasible at low cost and without difficulty; they are appreciably lighter and beyond comparison cheaper than framed steel girders of the same loading and span, especially in the case of several similar girders.

The Author has prepared a scheme, complete to the smallest detail of the constructional equipment involved, for girders of 100 m (328 ft.) span carrying a double roof (a saw-tooth roof over a transparent floor), the general lines of the idea being similar to those followed at Havre. The following results have emerged from this design:

The weight of the girder is 3200 kg per m run (2151 lbs. per ft. run). It supports its own weight, and an imposed load equal to its weight, by means of reinforcement weighing about 350 kg per m (235 lbs. per ft.) formed of 180 bars of 16 mm ($^5/_8$ in.) stretched to 84 kg per sq. mm (53 tons per sq. in.) and permanently tensioned to 50 kg per sq. mm (317 tons per sq. in.); the stress in the concrete is 180 kg per sq. cm (2560 lbs. per sq. in.) — which figure is admissible in view of the fact that the strength of the concrete is no less than 800 kg per sq. cm (11 380 lbs. per sq. in.). Provision has been made in the web for vertical tensile reinforcements and horizontal reinforcements as necessary, with a double connection ruling out any possibility of cracking due to shear. The thicknesses have been designed for a maximum shear stress of 60 kg per sq. cm (853 lbs. per sq. in.); under these conditions the shear forces do not cause any tension but they cause compression well below the accepted limit of 180 kg per sq. cm (2560 lbs. per sq. in.).

In conclusion, some details may be given of an application of these methods recently described by the Author for the improvement of concrete roads. The use in roads of reinforcement in the ordinary form often does more harm than good in that it aggravates the stresses due to shrinkage and expansion and so encourages instead of prevents crumbling and cracking under alternating loads. The use of stretched reinforcing bars, on the other hand, considerably improves the road construction from every point of view.

In the first place, these bars have the effect of substituting compression for the shrinkage stresses; thereby eliminating shrinkage cracks and the need for most of the joints — for any variations caused by combined changes of temperature and hygrometric condition are converted to mere variations in the intensity of compressive stress on either side of a mean value.

Secondly, the elimination of tension in the concrete brings about a considerable improvement in its resistance to abrasion; it reduces the deformability and increases the resistance to bending: hence good performance under heavy loads even in the case of a yielding subsoil.

Using steel at 120 kg per sq. mm (76 tons per sq. in.) the weight of reinforcement would be about 4—5 kg per sq. m (0,8 to 1,0 lbs. per sq. ft.) and its cost from 5 to 6 francs per sq. m. Hence the total increase in cost per sq. m would be equivalent to that of a few centimetres greater thickness but the improvement would be worth very much more than this greater thickness. Finally, the road could be handed over to the users two hours after placing the concrete.

The Author does not propose further to extend the list of possible applications. He is of opinion that systematic use of the hypotheses and methods of physics has been proved capable of greatly and rapidly advancing our general knowledge on the subject of cement and concrete, with corresponding benefit to the industries concerned in the application of these materials: indeed the progress made possible in this direction may well be comparable to that already achieved by the same means in the fields of metallurgy and mechanical engineering.

Summary.

Starting from a number of new assumptions, the Author surveys the general properties of cements and the formation of the network of pares in cement slurries. The strength of cements depends more on the mechanical and physical conditions of its application, than on any other factors. Very considerable, and almost instanteneous setting for all kinds of cement can be attained by simple improvements of the density.

IIb 4

Cracking in Reinforced Concrete.

Rißerscheinungen im Eisenbetonbau.

Fissurations dans le béton armé.

F. G. Thomas,
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The use in reinforced concrete construction of high tensile steel with increased working stresses must almost inevitably lead to increased cracking. In the past the cracking normally obtained had apparently no serious effect on the stability of the structure and was usually insufficient to cause important corrosion of the reinforcement. The cracking accompanying higher steel stresses, however, is proportionately greater than the increase in stress and it is possible that, with the higher stresses that could be allowed for high tensile steel on a yield point basis, the cracking will be more important.

The increased rapidity of hardening of modern cements is another factor which may seriously affect the cracking. Many complaints have been received at the Building Research Station recently of shrinkage cracking in cases where no trouble was previously experienced with the less rapid-hardening cements of 10—15 years ago.

In view of this tendency to increased cracking, therefore, work is being carried out under the supervision of Dr. *Glanville* at the Building Research Station, in co-operation with the Reinforced Concrete Association, to determine the factors that control cracking in reinforced concrete members. The present paper describes briefly some of this work.

Strain Capacity of Concrete.

It has long been thought that the criterion for failure of concrete in tension may be the ultimate tensile *strain* that can be sustained rather than an ultimate tensile strength. The strain capacity of concrete, i. e. the extension that can occur without the formation of cracks, has been determined by many investigators with widely different results. The lack of agreement is probably due to two main reasons: 1) variation in the initial stress conditions in the members before test and 2) variation in the accuracy of observing the appearance of cracks.

It appears that the effect of reinforcing steel is normally to increase the strain capacity of the concrete by only a small amount; when reinforced concrete members are stored in air, tensile stresses are set up in the concrete as a result of shrinkage, with an adverse effect on the subsequent strain capacity when the member is loaded. On the other hand, the presence of the reinforcement may lead to a considerable increase of the effective strain capacity under conditions of moist curing.

It is likely that the apparent increase in strain capacity due to the reinforcement observed by some workers was partly the result of insufficiently accurate observation of the appearance of the first crack. In tests at the Building Research Station, with a smooth whitened surface it has been found possible to detect cracks 0.0001 in. in width by eye, though normally the cracks are a little wider when they first appear. Crack widths are measured in all tests on reinforced concrete members, using portable microscopes with eyepiece scales.

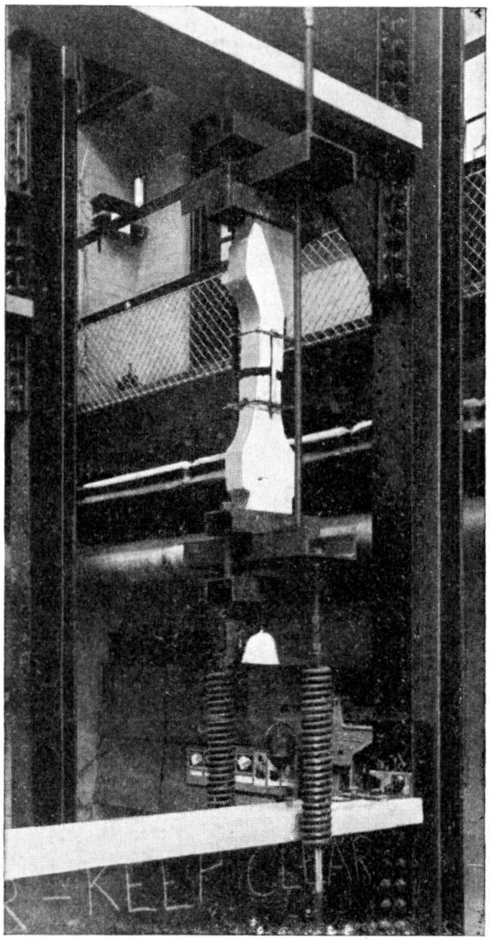

Fig. 1.
Measurement of Shrinkage Stresses in Restrained Concrete Members.

Shrinkage Cracking.

Shrinkage of concrete is probably the most frequent cause of cracking and also the most difficult to remedy or prevent. In the case of unrestrained reinforced concrete members, shrinkage induces tensile stresses in the concrete so that, particularly with high percentages of steel, cracking may occur even when no external load is applied. In practice, however, some degree of end restraint is almost always present, particularly in monolithic frameworks. The effect of creep of the concrete is to reduce the concrete stresses so that in this connection creep is helpful in reducing the tendency to crack. The resistance of a concrete to cracking for any particular end conditions can be obtained roughly from a study of the shrinkage, creep, elasticity and strength properties of the concrete, the combined effects being estimated mathematically. Although this method gives a fair comparative idea of the resistance to cracking there is sometimes doubt as to the exact creep properties of the concrete at stresses just below the ultimate tensile strength and a more direct experimental method has therefore been developed.

A special apparatus was designed in which concrete specimens, with extensometers attached to the central portion, were maintained under tensile loads by means of springs and these loads were adjusted periodically so that the shrinkage movements were entirely balanced by the elastic movements and creep due to

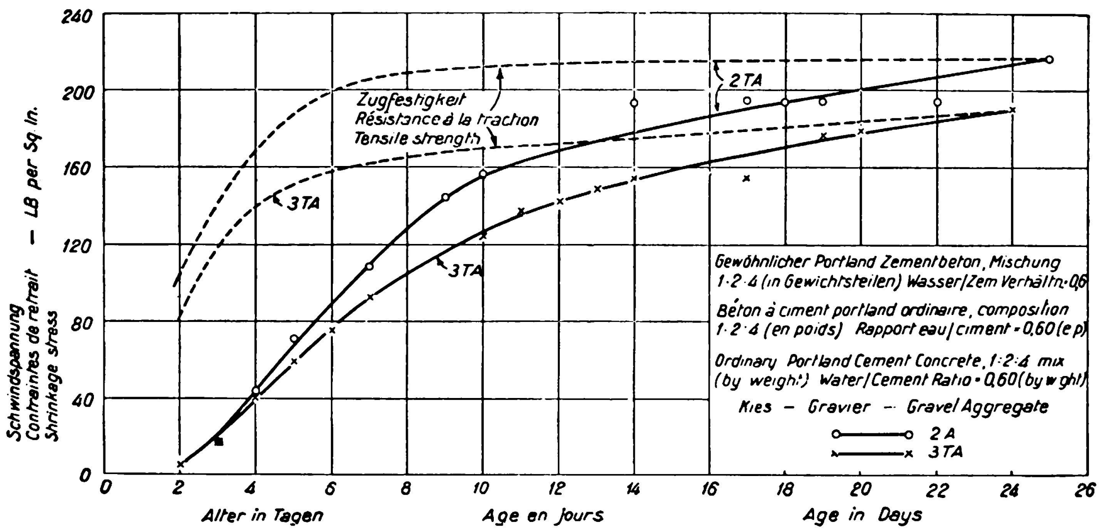

Fig. 2.

Resistance to Shrinkage cracking of completely restrained ordinary Portland Cement Concrete.

loading. By this means the actual shrinkage stresses set up in a completely restrained member could be measured. A photograph of the apparatus after failure of a specimen is shown in Fig. 1.

The results of duplicate tests on ordinary Portland, rapid-hardening Portland and high alumina cement concretes are shown in Fig. 2—4. A 1:2:4 mix (by weight) with a water/cement ratio of 0.60 was used in all cases. It will be seen from the figures that the shrinkage stresses are little different for the Portland cement concretes, but as failure is approached with ordinary Portland cement the development of stress decreases considerably as a result of large creep movements. This effect is not so great with rapid-hardening Portland cement, the stress increasing steadily until the tensile strength is reached and cracking occurs. With the high alumina cement concrete there is a rapid increase in stress, the factor of safety against cracking being negligible shortly after the commencement of the test.

Other tests have indicated that an increase of water content is not necessarily followed by a greater tendency to crack, and that the resistance to cracking is markedly affected by the type of aggregate used. It is realised that in practice complete restraint will not usually be imposed and that the relative resistances to cracking of various concretes may be somewhat altered by the degree of fixity. Further tests are therefore being made in which the end fixity is not complete.

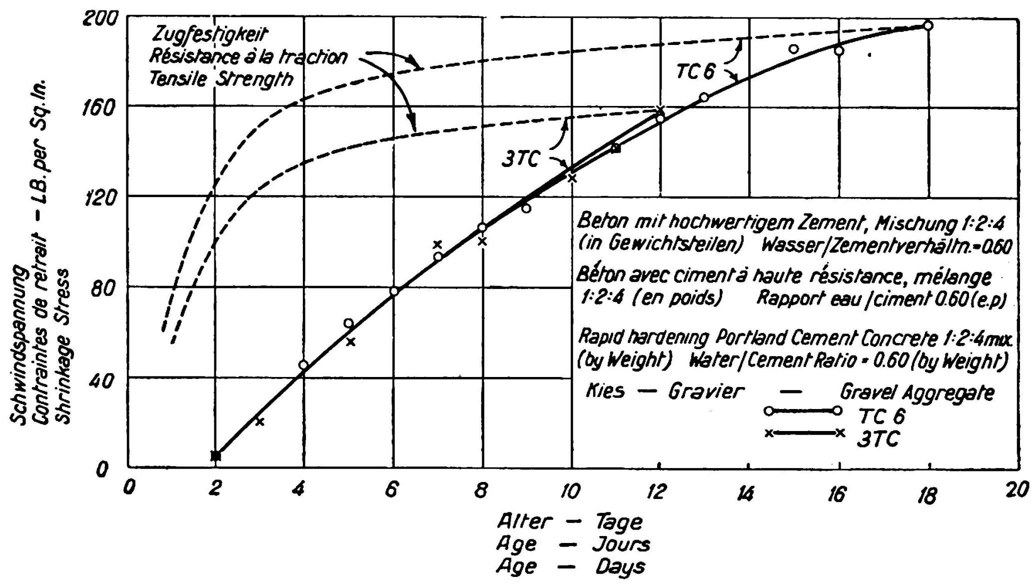

Fig. 3.

Resistance to cracking of completely restrained Rapid Hardening Portland Cement Concrete.

Strain Cracking.

In this section will be considered only the condition where the tensile forces producing cracking are the result of directly applied loading, as in the case of tests in bending or tension. In all tests on reinforced concrete members now being made at the Building Research Station the widths of cracks are measured and it is hoped that at a later date a complete analysis of these measurements will provide considerable information on strain cracking. Certain general results are already available and a few of the tests in which crack measurements have been made are described below.

In one test the beam was rectangular in section, 4 in. wide and $8^1/_4$ in. deep and 7 ft. long. The tensile reinforcement consisted of two mild steel bars of $^3/_8$ in. diameter at an effective depth of 7 in., and suitable stirrup reinforcement was provided to resist shear. The beam was loaded at third points on a 6 ft. span. The appearance and progress of cracks up the side of the beam were carefully observed and the widths of each crack near the bottom of the beam and at the level of the steel were measured for every increment of load.

The load was first increased steadily to the calculated working load and this load was maintained for 21 hours. After this period the load was gradually removed. The results of the crack measurements are shown in Fig. 5. It will be seen from this Figure that the total crack width (i. e. the sum of the widths of all cracks) increases to some extent during the period under sustained load, and that on removal of the load there is a partial recovery in crack width. It is interesting that on reducing the load slightly from the working value the total crack width increased somewhat, and one or two cracks increased slightly in length.

The recovery in crack width on removal of the load is what *Probst* calls the "elastic width" of the crack. In the present test the "elastic width" was on the average just over one-half of the actual width of crack. However, the term "elastic" must be used with caution for it is seen from Fig. 5 that the recovery does not take place uniformly on reduction of the load, but the crack width remains practically constant during the early stages of unloading. It is clear that

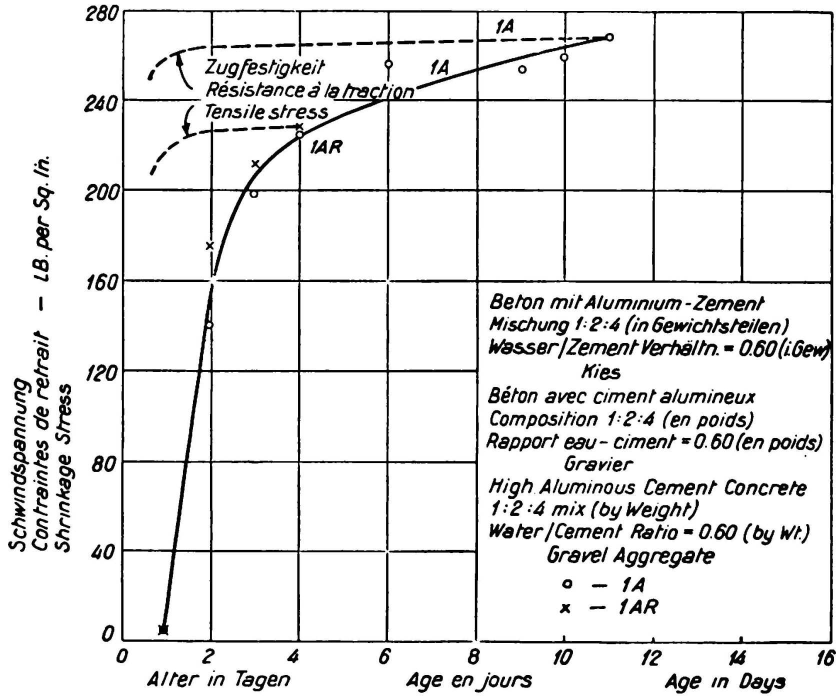

Fig. 4.

Resistance to cracking of completely restrained High Aluminous Cement Concrete.

before recovery can take place the slip mechanism at the steel has to be reversed and this reversal requires a substantial load change.

The beam was left without load only for sufficient time to measure the crack widths, and was then steadily loaded to a value of $1^1/_2$ times the working load. This load was sustained for 44 hours, during which period very little increase of cracking occurred. The load was then increased until failure of the beam resulted from yield of the steel.

In Fig. 6 the maximum crack width at the level of the steel has been compared with the steel stress, computed from the usual straight line-no tension theory. In this Figure the effects on removing the load have not, however, been included. It will be seen that there is a rough linear relationship between the crack width and the steel stress, and that the crack width is inappreciable on first loading until a stress of about 12,000 lb. per sq. in. The relationship is of the form expected from a simple analysis of the mechanics of cracking.[1]

The yield of the steel is shown in Fig. 6 by the very sharp bend in the curve at a stress of 47,000 lb. per sq. in. The deflection of the beam also increased rapidly at this load, but in cases where more than one layer of tensile reinforce-

[1] *Thomas, F. G.* „Cracking in Reinforced Concrete". Struct. Eng. 1936, 14 (7), 298—320.

ment is used it has been found that the crack widths are a much better guide to the yield point of the steel than the deflection. This point was shown up well in some recent tests at the Building Research Station on two-span continuous beams. The results for one test are shown in Fig. 7. From the crack width diagrams it is seen that the yield point loads at both the central support and in the span are quite clearly defined, but there is no definite indication of the value of these loads from the deflection curve. It is seen, therefore, that in such tests the measurement of the crack widths will help materially in the analysis of the test.

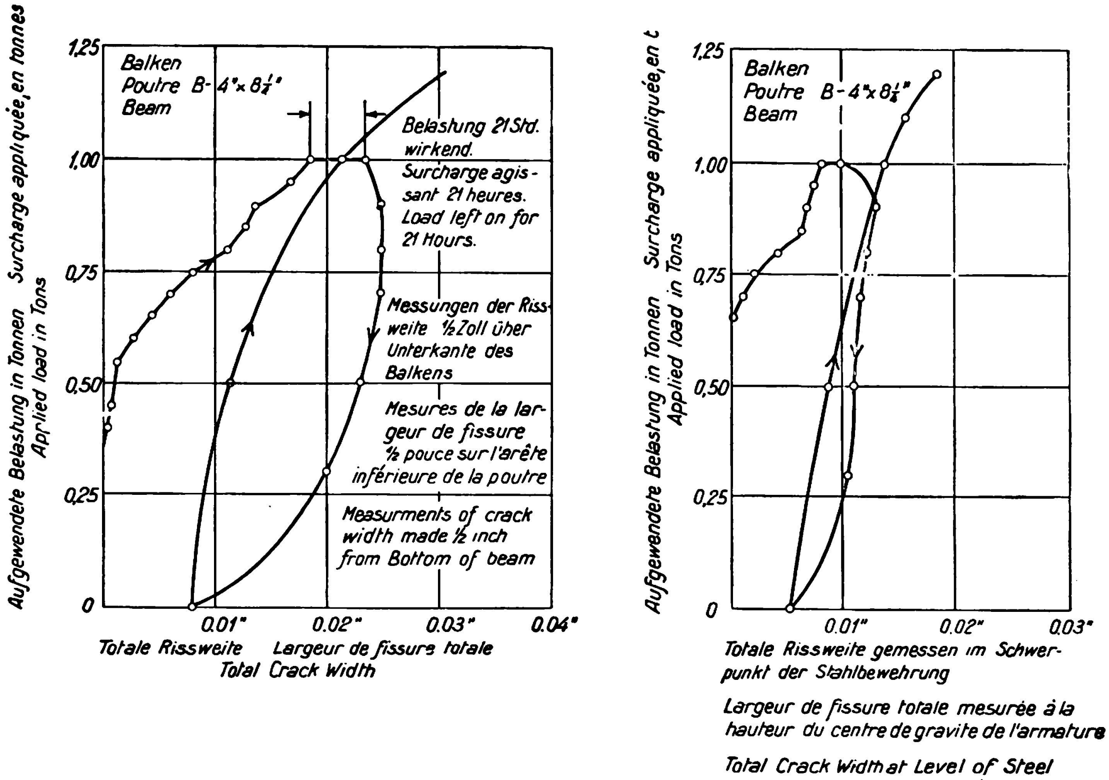

Fig. 5.

Recovery of cracks.

The effect of the percentage of the steel on the crack width when the bar size is kept constant was investigated in some tests on a high tensile steel. Ten beams were tested, all of length 9 ft. 6 in. and overall depth $10^5/_8$ in. Five different widths were used varying from $6^1/_4$ in. to $14^1/_2$ in., two beams of each width being tested. The tension reinforcement consisted in all cases of two compound bars, each of which was made up of two $^1/_2$ in. diameter round bars twisted together helically; the percentage of steel varied, therefore, from about 0.6 to 1.4.

The beams were tested by loading at two points symmetrically 2 ft. 6 in. apart on a span of 9 ft. The results of the measurements of the crack widths in the parts of the beams under constant bending moment (no shear) were as follows:

1) The relationship between steel stress and crack width is not wholly linear. The probable reason for this is that the slope of the stress-strain curve for steel is not constant for the high tensile steel tested but decreases at high stresses.

2) In general, if the crack width-steel stress curve is produced to cut the steel stress axis, the extrapolated stress for zero crack width increases as the percentage of steel decreases. The values for this stress were:

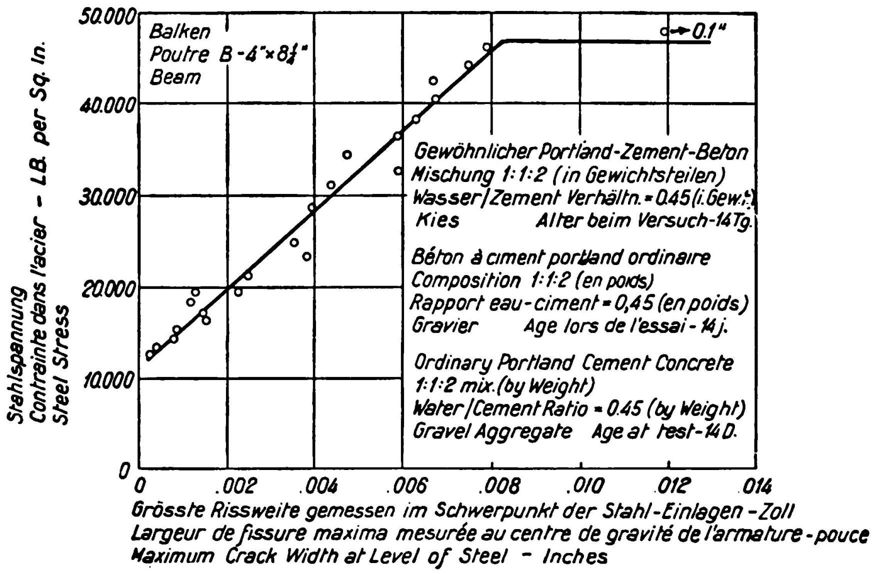

Fig. 6.

Dependence of maximum crack with on steel stress.

Percentage of steel	1.38	1.19	0.98	0.78	0.59
Steel stress for zero crack width — lb. per sq. in.	5,900	4,600	8,900	10,000	13,500.

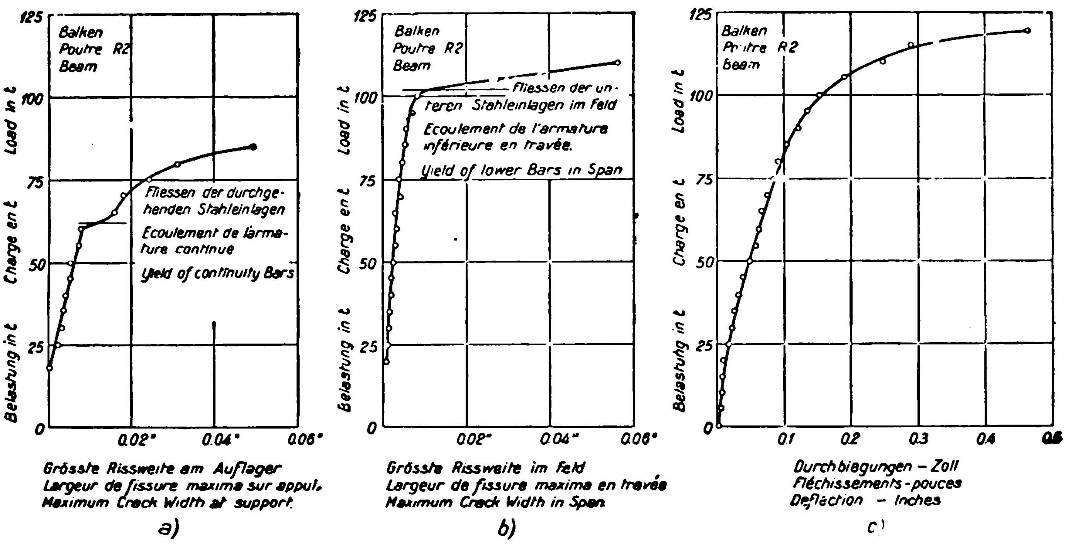

Fig. 7.

Maximum cracks width as guide to steel yield.

This effect tends to keep the cracks small at working stresses for low percentages of steel.

3) The rate of increase of crack width with stress is greater for low percentages of steel. The increases in crack width between 18,000 and 40,000 lb. per sq. in. were:

Percentage of steel . . .	1.38	1.19	0.98	0.78	0.59
Increase in crack width — in. $\times 10^{-3}$	3.9	5.2	6.4	7.3	8.1.

This effect tends to make the cracks bigger for low percentages of steel, particularly when the stresses are increased beyond the values now used in design. If, therefore, for high tensile steel higher steel stresses are allowed leading to decreased percentages, the cracking may be increased as a result of both the increased stress and the decreased percentage if the bar size is unaltered. It should be remembered at the same time that the percentage increase of cracking is decidedly greater than the percentage increase in working stress in the steel.

Effect of Prolonged Loading on Cracking.

An increase with time of the widths of cracks may result from two effects; first, the increase in steel stress due to a continuous breakdown of the concrete in tension and to the creep of the concrete; and second, a creep in bond causing increased slip of the concrete along the steel away from the crack.

Measurements have been made at the Building Research Station of the increase of cracking in reinforced concrete beams, and in one series of tests four beams were maintained under prolonged loading. High tensile steel was used for two of the beams and ordinary mild steel used in the others. At an age of 12—13 days the beams were loaded so that the theoretical maximum steel stress was 20,000 lb. per sq. in. for the plain bars and 27,000 lb. per sq. in. for the high tensile bars. The load was maintained for 6 weeks and was then altered so that the theoretical steel stresses were increased by 50 per cent. This load was sustained for a further period of 6 weeks before the beams were tested to destruction. It was found that the crack widths increased by about 50 per cent. during the early stages of the test when the cracks were extending up the sides of the beams and the concrete creep was comparatively large; and that at a later age, even at the higher steel stresses. the change in crack width with time was small.

Pretensioning of the Reinforcement as a Preventative of Cracking.

The possibility of preventing cracking at working loads by artificially obtaining an initial concrete compression has frequently been advocated, notably by *Freyssinet.* The method is sometimes applied in the case of precast concrete floor slabs. The tension reinforcement is stressed to a high percentage of its yield strength by means of springs or levers before the concrete is cast. The pretensioning apparatus is left in position until the concrete has sufficiently hardened to take the stresses induced in it when the forces in the reinforcing bars are allowed to be taken up by adhesion between concrete and steel.

There are, however, certain difficulties. Immediately the steel load is transferred from the pretensioning apparatus to the concrete section there is a compressive strain in the concrete resulting in a release of tensile load in the steel. At the same time there will be slip at the ends of the bars over the distance required to develop the maximum steel stress, and it would be wise, therefore, to delay the removal of the pretensioning device until the bond strength is sufficient to reduce this distance to a fraction of the whole length of the slab.

Further, between the time of removing the pretensioning apparatus and applying the working load the concrete continues to deform as a result of the creep of the concrete under the action of the internal load, and also through the normal shrinkage of the concrete. Since shrinkage tends to decrease the strain capacity of all air cured concrete this factor does not enter into a comparison between members with and without pretensioning, but in calculating the pretension necessary to prevent cracking under a given bending moment the effect of shrinkage must certainly be considered.

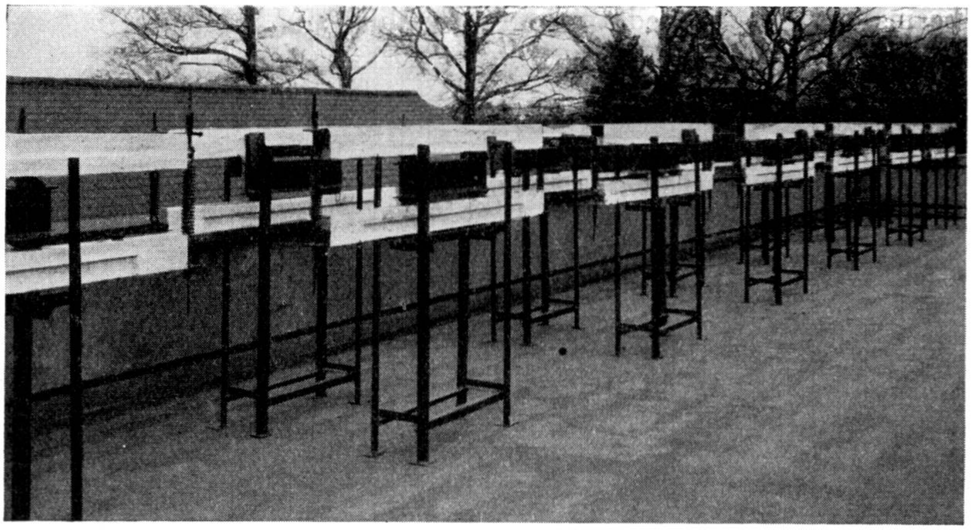

Fig. 8.
Exposure Tests on Reinforced Concrete Beams.

The results of tests at the Building Research Station to determine the effect of pretensioning the tension reinforcement of beams made with foamed slag concrete are given below. The beams were 6 ft. long and of rectangular section, $4^1/_4$ in. wide by $6^1/_2$ in. deep, with two $^1/_4$ in. diameter high tensile steel bars as tension reinforcement and two $^1/_4$ in. diameter mild steel bars as compression reinforcement. The concretes used were:

Beams PT 1 and PT 2. Rapid-hardening Portland cement concrete, $1:1^1/_4:1^3/_4$ by volume, 1:0.55:0.54 by weight, water/cement ratio 0.53 by weight with foamed slag aggregate of maximum size $^3/_{16}$ in.

Beams PT 3 and PT 4. As above, except that the proportions were $1:2^1/_2:3^1/_2$ by volume, 1:1.10:1.09 by weight, water/cement ratio 0.80 by weight.

The tension bars of beams PT 1 and PT 4 only were stressed to an initial tension of 40,000 lb. per sq. in. before placing the concrete, and the pretensioning apparatus was left in position until an age of 14 days. All specimens were stored under damp sacks for 4 days and subsequently in air at 64^0 F. and 64 per cent. relative humidity. At an age of 14 days the pretensioning device was removed from the two beams so that the steel load was transferred to the concrete. All beams were tested at an age of 28 days by loading at third points on a 5 ft. span. A very high tensile steel was used, the failing strength being 120,000 lb. per sq. in. (based on original area); there was no clearly defined yield point but the stress corresponding to a permanent deformation of 0.2 per cent. was 100,000 lb. per sq. in.

The main results of the tests are given in Table I. At a steel stress of 25,000 lb. per sq. in., calculated according to the usual no-tension theory, there were no cracks in the pretensioned beams but those in the other beams had reached widths of 0.003 and 0.005 in. The deflection at this stress was reduced as a result of the pretension to $^1/_3$ and $^1/_4$ of the deflection of the beams without pretension. In order to obtain the same deflection and crack widths as those that were in the beams without pretension at a steel stress of 25,000 lb. per sq. in. the pretensioned beams had to be loaded to about twice this value.

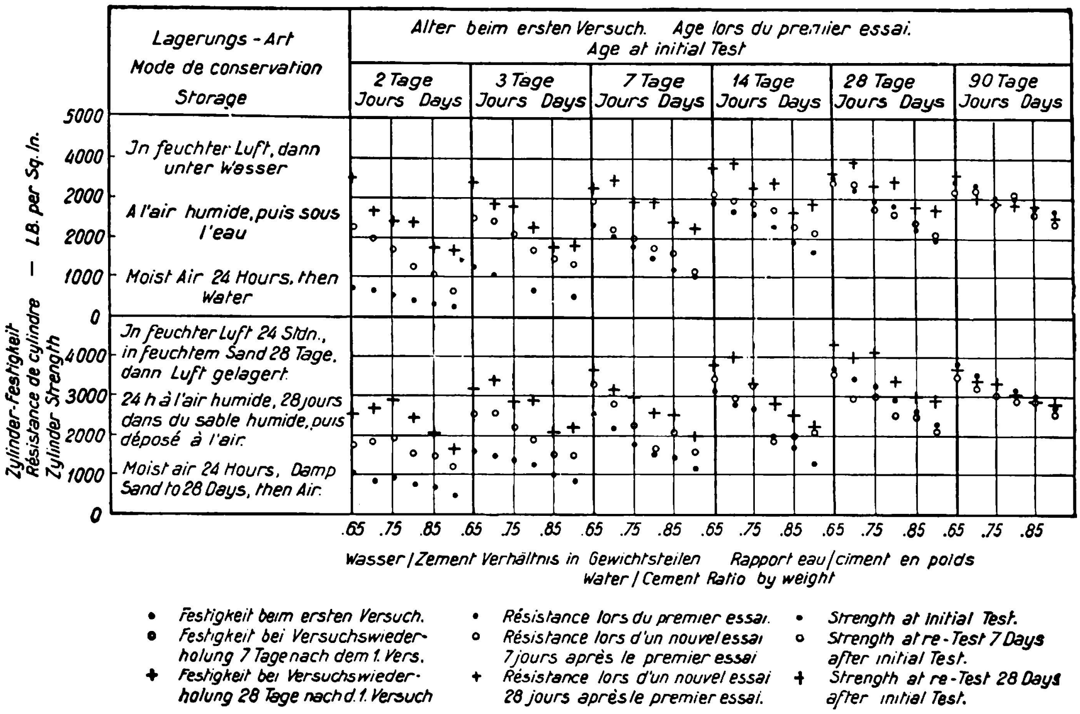

Fig. 9.

Autogenous Healing of concrete.

It is clear, therefore, that pretensioning of the reinforcement is extremely useful in reducing deflection and cracking. It will be noticed, however, that the increase in stress to give the same conditions as in the beams without pretension was about 25,000 lb. per sq. in., and not 40,000 lb. per sq. in., which was the original pretension applied to the steel. Less than two-thirds of the nominal pretension was effective at the time of loading. The reason for this is that the initial strain in the steel is reduced as a result of concrete deformation when the load is transferred from the pretensioning apparatus to the concrete and by subsequent creep of the concrete. The pretensioning had no effect on the failing load of the beams.

TABLE 1.

Effect of Pretensioning Reinforcement of Beams.

	Beam No.			
	PT_1 [1]	PT_2	PT_4 [1]	PT_3
Crack width at steel stress, due to external load, of 25,000 lb. per sq. in. — inch.	0	0,003	0	0,005
Steel stress at which cracking started — lb./sq. in.	35,000	17,000	35,000	14,000
Steel stress in pretensioned beam to give same crack width as in beam without pretension at 25,000 lb./sq. in. — lb./sq. in.	55,000	—	52,000	—
Deflection at mid span at steel stress of 25,000 lb./sq. in — inch	0.019	0.054	0.018	0.080
Steel stress in pretensioned beam to give same deflection as in beam without pretension at 25,000 lb./sq. in. — lb./sq. in	48,000	—	47,000	—
Weight of concrete per cubic foot. — lb.	115		110	
Concrete strength (4 in. cube) — lb./sq. in.	4900		3800	
Average bond strength obtained with a 1/4 in. high tensile steel bar embedded in concrete cylinder, 3 in. diameter, 6 in. long. — lb./sq. in.	14 days 330 28 days 320		310 350	

[1] With tension steel pretensioned to 40,000 lb. per sq. in. (nominal).

Corrosion.

It has been suggested that there is a limiting width of crack below which corrosion of the reinforcement will not take place. Although this seems reasonable, satisfactory evidence on this point has yet to be obtained. It is felt that the best way to do this is by actual exposure tests on loaded reinforced concrete beams, and such tests have been started at the Building Research Station, measurements being made of the progressive cracking of the beams. A photograph of some of the beams is given in Fig. 8. The usefulness of exposure tests by certain other investigators has been severely restricted owing to the lack of data with regard to the widths of the cracks.

Healing of Cracks.

A few years ago Professor *Duff Abrams*[2] tested a number of concrete cylinders to failure, and retested them after a period of some years; they not only took as much load as they had originally taken but gave values from 167—379 per cent. of the original 28-day strength. *Abrams'* opinion was that the small cracks

[2] *Abrams, D. A.* „Question Box". Am. Conc. Inst. Proc. 1926. 22, 636—9.

which opened up at the time of the original test were actually welded together by the subsequent depositing of the soluble materials from the cement and aggregate. It was actually a healing process and the concrete gained in strength much as it would if it had not been subjected to its original load.

Tests carried out at the Building Research Station have confirmed the results of *Abrams*. The tests were on 8 in. × 4 in. cylinders which were retested at quite short periods of either water, damp sand or air storage after loading to failure. The cylinders were tested in a hydraulic testing machine and were not shattered under test. The results of a few of the tests which may be regarded as typical are given in Fig. 9. In this Figure, which relates to Portland cement concretes of various consistencies, the strengths obtained on first testing are compared with the strengths on re-testing after a period of 7 days and again at 28 days from the initial test. From this Figure it will be seen that the healing is greater for concrete initially crushed at early ages than for older concrete. In most cases a period of only 7 days is sufficient for the concrete to heal sufficiently to bear at least the load that caused failure originally and only in the case of concrete initially tested at 90 days is the period of 28 days insufficient, and even here the difference in ultimate strengths is not very much.

Similar results were obtained with several batches of cement including aluminous cement. In general it was found that:

a) the leaner and more permeable the mix the greater the amount of healing and b) the wetter the mix the greater the amount of healing.

Summary.

A method has been developed whereby the shrinkage stresses in restrained concrete members can be measured until cracking occurs. It has been shown that there is tendency for the resistance to cracking to decrease as the rapidity of hardening of the cement used increases.

The suggestion put forward by several investigators that cracks are to some extent "elastic" — that is, they recover somewhat when the load is removed — has been confirmed, but it is clear that the term "elastic" is not very satisfactory. The cracks do recover when the load is completely removed but the recovery is not proportional to the reduction in load. In fact, a reduction of one-half of the load may cause no change at all in the crack widths owing to the hysteresis due to the change in direction of the slip mechanism at the steel-concrete interface.

It has been found that for a particular bar size the crack widths increase with steel stress more rapidly for low percentages of steel. The increase in crack width that would result from an increase in the working stresses in the tension steel may proportionately be much greater than the increase in stress, particularly as the percentages of steel normally used would tend to be reduced.

Considerable development of cracking may occur in beams under prolonged loading, though a state of equilibrium is reached after a few weeks from loading.

Tests in which an initial pretension of 40,000 lb. per sq. in. was applied to the tension steel of beams have shown that the effect of the elastic and inelastic movements of the concrete may reduce appreciably the effectiveness of pretensioning. In the particular tests cited, the pretensioning apparatus was removed at an age of 14 days, and the beams were loaded at an age of 28 days. The effective pretension had during the intervening period been reduced by concrete deformation to only about two-thirds of its original value.

A series of tests has been made which indicated that fine cracks in concrete members often heal completely with time. The healing process takes place to some extent in air but is more complete in moist curing conditions.

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