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Investigations concerning the co-operation of old and newly added concrete structural parts

Versuche zur Bestimmung des Zusammenwirkens von altem und neuem Beton

Recherches sur la collaboration d'ancien et de nouveau béton

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How far the original substances of a destroyed concrete-construction co-operate with the newly added parts of concrete has not yet been sufficiently examined. Systematic research is still lacking. Theoretical considerations which are not based on a sufficiently great number of investigations are not enough to produce a satisfactory solution.

In spite of the enormous number of demolished structures which were left nearly everywhere in Europe after the war, there are still damaged structures which can be saved through adding new parts. We will now examine how the two parts co-operate and if a satisfactory composite structure is possible.

Even if concrete with the same aggregate and with the same quantity of cement were utilized, the physical properties of the old and new substance will be different. It is questionable whether a real co-operation could be obtained under such circumstances. It happens that the reinforcement stands out of the structural parts remaining and is cemented in the newly built parts; these reinforcements are often greatly deformed. This deformation is beyond the yield point and the influence of this deformation on the carrying capacity of the new composite construction is yet an unsolved problem. Often in the old parts of the construction there are very thin, nearly invisible cracks caused by explosions, percussions and the like. Even these kinds of cracks have influence, not yet examined, on the carrying capacity, shearing and tensile strength, and deflexion of the structural parts. Here, of the very rare literature I would mention only Ing. Herman Goebel's article ,,Der Wiederaufbau des Ammoniakwerkes Oppau'', Bauingenieur, 1923, Number 12.

Ing. Goebel discusses in detail the work of reconstruction using the old structural parts. But no research in laboratories or otherwise was made concerning the co-operation of the newly joined construction. Also there are no published memoranda as to what extent – althoug the effective load was reduced to half – this co-operation ceased working, after years, in the heavy industrial structures and silos.

To clear up this problem, I made the following investigations in the Science of Mining Laboratory of the Mining Academy in Cracow.

We had to examine:

reinforced concrete girders damaged by flexure, and
reinforced concrete pillars damaged by cracks.

I. Test of flexure on girders

There were four series of test made. Each consists of 5 girders. In order to avoid for the time being too many complications in solving this problem, all of the 20 girders were made of the same length and of the same dimension of transverse-section. To construct the girders and pillars the same aggregate (quantity and quality), the same kind of cement (quantity and quality) and the same proportion of water were used, to avoid the influence of different kinds of concrete. Also the influence of different temperatures was avoided. Although the tests were made during the winter and partly in spring, a nearly constant temperature of about 16 °C was secured in the test-room.

The full length of the girders was 1,10 m (43,3 in.) and the dimension of the transverse-section 10 · 16 cm (3,9 · 6,3 in.). There were 4 series constructed each of 5 girders. Each of the series consisted of:

one full length girder without joint
one girder of 0,2 of the full length
„ „ „ 0,4 „ „ „ „
„ „ „ 0,6 „ „ „ „
„ „ „ 0,8 „ „ „ „

The shorter girders were later (after weeks or months) completed to full length with fresh concrete.

The first group of girders (No. 1, 2, 3, 4, 5) was reinforced with 4 pieces 5 mm (0,2 in.) standard steel without stirrups.

The second group (No. 6, 7, 8, 9, 10) has the same reinforcement, but with additional stirrups.

The third group (No. 11, 12, 13, 14, 15,) was reinforced with 8 mm (0,31 in.) steel with two 5 mm (0,2 in.) stirrups.

The fourth group (No. 16, 17, 18, 19, 20) was taken from a slab of reinforced concrete destroyed through bombing.

	constructed	added	difference	destroyed after
Series 1	12. XII. 1947	11. III. 1948	88 days	126 days
Series 2	18. XII. 1947	12. III. 1948	83 „	120 „
Series 3	9. I. 1948	13. III. 1948	63 „	93 „
Series 4	3. VI. 1948	2. VII. 1948	28 „	70 „

The enclosed schedules show at what time the several girders were constructed and at what time the new concrete was added. Also the time at which tests were made to destruction is shown on the schedules. In reality of course the time which passes between the construction of the old and newly added parts is much longer (sometimes even many years). Because of the comparatively small number of tests, I had to be content to establish the influence of time elapsing between the two constructions joined with new concrete.

I wanted to establish the distinct reaction of the different kinds of girders through systematic bending tests concerning flexure, made under the same conditions. These tests were based on experiments which I made years ago, i.e. the girders constructed with the same cement, with the same admixture of substance (concerning quality and quantity) and the same quantity of water show a considerable smaller increase with flexibility than with the ability to resist load. The flexibility depends functionally on the whole structure of the girder, while the fracture (and therefore also the ability to resist load) results in consequence of structural, often local, defects.

The flexibility was established at the points α β γ and from the flexibility of the concrete girders the E (modulus of elasticity) was calculated. On the strength of the usual ferro-concrete theory the theoretical flexibility was calculated from the dimensions and reinforcement, and plotted out on the drawing.

I want to say the following concerning the calculation of the modulus of elasticity: it is not to be confounded with the usual conception of the elastic-modulus of concrete which is by no means a clear conception, with only one meaning. Concerning the elementary theory of the ferro-concrete calculation its application leaves much to be desired.

To simplify the manner of treatment of the deflexion of the girder, we neglect for a while the composite nature of the material, treating it as if it consisted of single substance. Calculating the moment of inertia we have to take into consideration the transverse-section of steel and the modular ratio as 15, and we employ calculating the theoretical deflexion the deflexion-formulae which are based on Hugh's and Bernoulli-Navier's hypothesis (schedule).

We derive the E from the deflexion-formula. This is the elasticity-modulus of an ideal girder, the flexibility of which, when loaded equally, is the same as that of our reinforced test girders.

We neglect the tension which results after the deflexion. We shall solely establish and afterwards compare the actual deflexion. All our conclusions are derived from these comparisons. We avoid in this way all the contradictory assumptions of the basic ferro-concrete theory; as: the linear stretching, the remaining of the plane nature of the surface of transverse-sections, the constant elasticity modulus of concrete and steel. If in spite of that and taking into consideration every kind of loading we calculate and trace out

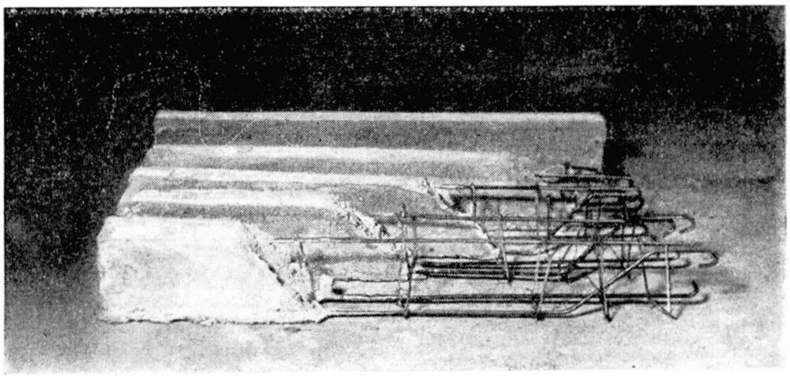


Fig. 1

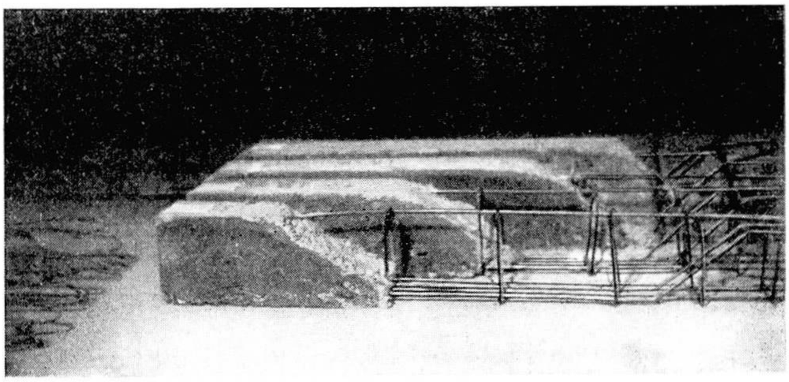


Fig. 2

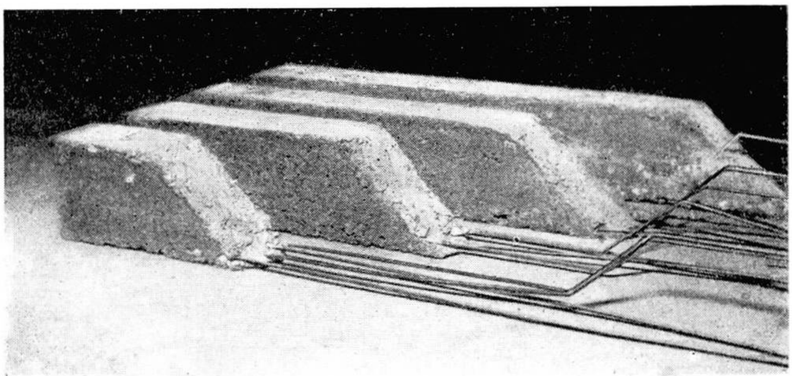


Fig. 3

our E-quantity derived from the theoretical deflexion-comparison, we obtain a very interesting and very clear picture of the newly added construction reacts.

To obtain a better criterion we put on two coherent schedules the flexibility and the comparison of the theoretical deflexion of all the 20 girders. We traced out the transverse-section and loading values and the calculation of the theoretical bending and the theoretical stretching. The latter have no particular influence on the further treatment of this problem.

						b = 11 cm h = I l = 110 cm					m = 297 cm			$a=\frac{1}{2}(l-20)=41{,}25$	
						l_o = 102,5 cm					n = 9,5 cm			$x=\frac{1}{2}(l-45)=28{,}75$	
	G	m	Q	ΣQ	P						f_α		f_β	f_γ	
						LD	SD	PD	LG	PG		teor.		teor.	
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	I NP 16 53,163	148,5	831,02	831,02	415,51	0,283 0,300	0,432 0,465	0,387 0,435	0,000684 0,000	0,065	0,283	0,690	0,387	0,989	0,432
2	+7,25	289,0	220,55	1051,57	525,79	0,471 0,500	0,725 0,780	0,569 0,650	0,00116 0,000	0,110	0,471	0,873	0,569	1,251	0,725
3	+7,25	289,0	220,55	1272,12	636,06	0,668 0,710	1,070 1,150	0,802 0,920	0,00168 0,000	0,160	0,668	1,056	0,802	1,514	1,070
4	−2·7,25	300,0	441,10	831,02	415,51	0,610 0,660	0,960 1,055	0,685 0,825	0,00200 0,000	0,190	0,610	0,690	0,685	0,989	0,960
5	+25,0	289,0	789,47	1620,49	810,25	1,026 1,115	1,667 1,810	1,129 1,325	0,00237 0,030	0,255	1,026	1,345	1,129	1,928	1,667
6	+7,25	289,0	220,55	1841,04	920,52	1,205 1,325	1,967 2,150	1,330 1,575	0,00279 0,050	0,315	1,205	1,528	1,330	2,191	1,967
7	5''	–	–	–	–	1,189 1,350	1,975 2,190	1,381 1,650	0,00242 0,100	0,330	1,189	1,528	1,381	2,191	1,975
8	+7,25	289,0	220,55	2061,59	1030,80	1,349 1,540	2,295 2,550	1,576 1,895	0,00284 0,120	0,390	1,349	1,711	1,576	2,453	2,295

$E = 140\,000$ kg/cm^3	$a^2 = 1701{,}56$	$\alpha = \beta = \dfrac{X}{6EJ}\left[3(a^3+ab) - x_1^2\right] = 0{,}000\,000\,0246 \cdot 6753{,}12 = 0{,}000\,166 \cdot 10$
$J_x = 1392{,}60$ cm^4	$x_1^2 = 826{,}56$	$\gamma = \dfrac{a}{24EJ}(3l^2 - 4a^2) = 0{,}000\,000\ \ 009\,62 \cdot 24\,712{,}51 = 0{,}000\,238 \cdot 10$
$W_{xb} = 296{,}60$ cm^3 $W_{xz} = 9{,}72$ cm^3	$l^2 = 10\,506{,}25$	

	P	M	$\sigma_{\dot z}$ / σ_b	f_α	f_β	f_γ	$\alpha'=\beta'$	γ'	E'_α	E'_β	E'_γ
0	1	2	3	4	5	6	7	8	9	10	11
1	415,51	17 139,79	1763,4 57,8	0,283	0,387	0,432			34 107	14 941	29 240
2	525,79	21 688,84	2231,4 73,1	0,471	0,569	0,725			25 932	21 466	22 047
3	636,06	26 237,48	2699,3 83,5	0,668	0,802	1,070			22 119	18 424	18 071
4	415,51	17 139,79	1763,4 57,8	0,610	0,685	0,960	23,23	30,40	15 823	14 091	13 158
5	810,25	33 422,81	3438,6 112,7	1,026	1,129	1,667			18 345	16 671	14 776
6	920,52	37 971,15	3906,5 128,0	1,205	1,330	1,967			17 746	16 078	14 227
7	920,52	37 971,15	3906,5 128,0	1,189	1,381	1,975			17 985	15 484	14 169
8	1020,80	42 520,50	4374,5 143,4	1,349	1,576	2,295			17 951	15 194	13 654

$$M = P\cdot a \quad a=41{,}25\,\text{cm} \quad \alpha'=\beta'=\frac{x_1}{6j}\left[(3a^2+ab)-x^2\right]=0{,}003\,44\cdot 6753{,}12=23{,}23$$

$$\sigma_b = \frac{M}{W_b} \quad W_b = 296{,}60\,\text{cm}^3 \quad \gamma'=\frac{a}{24j}(3l^2-4a^2) = 0{,}001\,23\cdot 24\,712{,}51 = 30{,}40$$

$$\sigma_{\dot z} = \frac{M}{W_{\dot z}} \quad W_{\dot z}= 9{,}78\text{cm}^3 \quad \frac{x_1}{6J} = \frac{28{,}75}{6\cdot 1392{,}60} = 0{,}003\,44 \quad \left[(3a^2+ab)-x^2\right] = 6753{,}12$$

$$E'_\alpha = E'_\beta = \frac{P}{f_\alpha}\cdot\alpha' \quad \frac{a}{24j} = \frac{41{,}25}{24\cdot 1392{,}60} = 0{,}001\,23 \quad (3l^2-aa^2) = 24\,712{,}51$$

$$E'_\gamma = \frac{P}{f_\gamma}\cdot\gamma \quad x_1=28{,}75\,\text{cm}$$

The bending was effected through a mechanism of levers in the ratio of 1 : 20 and measured with the aid of 5 Zeiss-dials for measuring at the points $\alpha\ \beta\ \gamma$ (3 measurements) and at the end of the girders (2 measurements). The dials can be read accurately to $^1/_{100}$ mm.

Comparison of the deflexion

In the 6 graphic schedules three demonstrate the deflexion of the girders at the points α and β, also at points where the symmetric loads have their effects; the other three demonstrate the deflexion of the girders at the central points. On each schedule 5 girders of equal reinforcement are dealt with. One of them is homogeneously built of concrete, the four others in partly new and partly old concrete with junctions at a different distance. For each of the groups the theoretical deflexion-line was calculated and traced out. It is known that the deflexion-line of a certain point is a straight one, i..e. that the deflexion of a point is theoretically proportional to the load. In practice there generally is a great discrepancy concerning this linear relation. The deflexion-lines for the points α and β should be theoretically the same. But this is not so, not even if homogeneously built. The disparity of deflexion is probably due to structural want of symmetry. The deflexion-lines of the points α and β and the deflexion-line of the point γ are not parallel. The γ lines generally are steeper, the moment and stress greater, the fractures show earlier and the deflexion-line reaches earlier the plastic sphere than at α and β - lines.

On a plain girder there is generally (but not always) the smallest deflexion to be seen. The cracks in it are nearly always later shown and the plastic sphere is reached at the latest time.

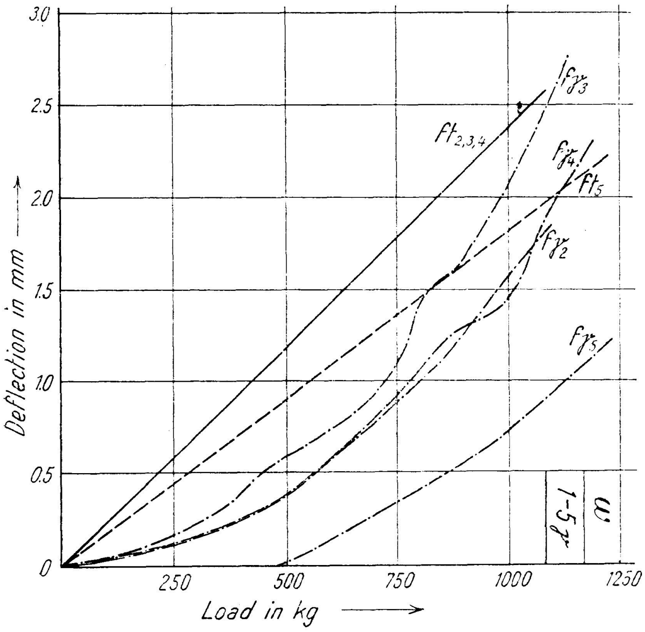
The deflexion-lines of the girders newly joined with concrete, are variable and more sensitive. They have not the same slightly parabolic character as those of the plain girders. One has the feeling as if on the newly joined parts on the deflexion internal stretchings are levelled down in wave-like step, similar to the re-crystallization of cast-iron bars. The concrete girders built in two stages which, as above mentioned, reach the plastic sphere earlier, have a smaller load capacity.

A functional connexion between the distance of the junctions and the deflexion-dimension can hardly be established. But it is almost true, that the biggest deflexion arise on girders, on which the junctions are in the center of the girder with some exceptions. Between the deflexion of girders without stirrups and those with stirrups there seems to be no great difference and the stirrups seem to have no greater tension-distributional influence concerning the deflexion.

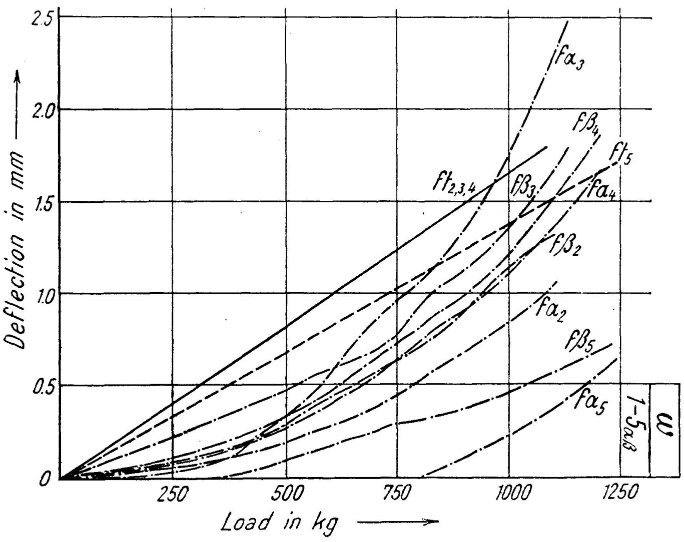
In the first and second group (of 4 five-millimetre reinforcement) the bending lines contrary to the third group (of 2 eight-millimetre reinforcement), also with a greater reinforcement cross-section, show a quiet course. The four round bar-reinforcements seem to equilibrate the internal tension better than the 2 with a greater cross-section.

Concerning the last group (No. 16, 17, 18, 19, 20) it had to be established how the reinforcement taken from shattered constructions works in the newly joined concrete. The concrete-girders made from destroyed slabs, are made on the same principle as the rest (16, 17, 18, 19), all the four of them of reinforcement taken from shattered constructions. The girder No. 20 is also made of concrete and reinforced with steel. The deflexion-lines of the points α β and γ show clearly and without doubt the influence of the steel-reinforcement which is loaded to the utmost limit of bending. This fact is distinctly visible in girders 18 and 19. The α β and γ lines show great variation even after a relatively small loading. Thus the old reinforcements remaining from the shattered constructions have a great influence upon the lessening of the carrying capacity of the composite construction.

As above mentioned our E-values are in no simple connection with the moduli of elasticity of the concrete and steel; thus, even if there were a formal connexion to be found, I do not consider that the efforts made in this respect clear up our problem. The calculation of our E-value derived from the deflexion-value also contains physical hypotheses which do not entirely agree. All the E-curved lines have a common characteristic feature: they have a parabolic form of greater importance and approach the axis of forces asymptotically. The E-value diminishes with greater loading. It is clear that if near the plastic sphere the load is heavier, the deformations are greater. The E-value is on the contrary proportional to the deformation. The E-curve lines are in the same girder nearly parallel concerning the points α β and γ. With one part concrete (girder No. 10) the three lines nearly synchronize with each other. Also there where the points of measurement α β and γ fall on the concrete built at the time (the three of them on the older, or the three of them on the newer part), the curve-lines are parallel and very near to each other. But if the points lie on different parts (new and old) the E-curve-lines not only are not parallel but sometimes cut each other and have irregular local deviation. The E-curve-lines are very sensitive to all the internal tensions which arise in the joined construction through additional building with con-

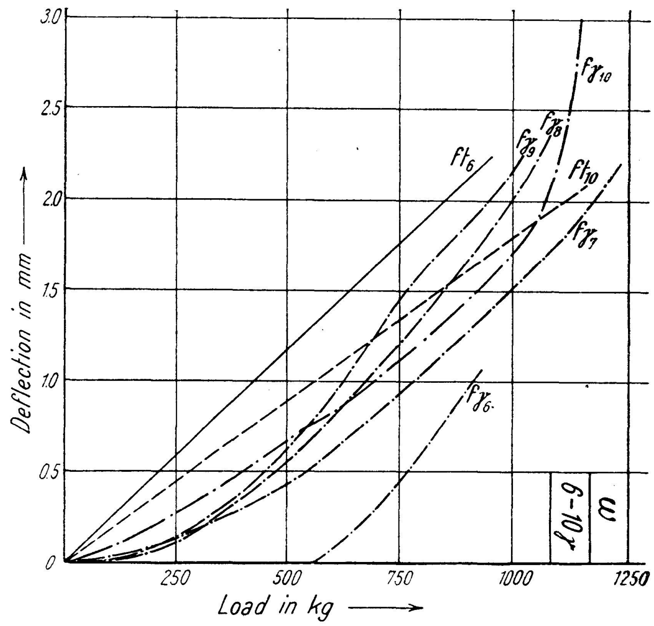


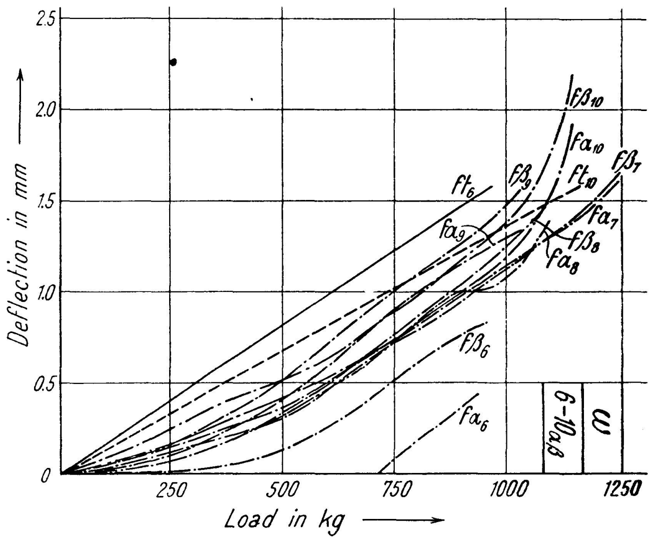
Comparison of the deflection



crete. The biggest differences generally are in the smallest deflexions and slowly diminish in the plastic sphere. The analysis of the curve-lines of the girders No. 16 to 19 confirm the result obtained of the deflexion-line. It is clear also here that the reinforcement works even under a smaller load already in the plastic sphere, thus explaining the early proceeding of cracks in the concrete. The E-curve-line of girder No. 20 (one part concrete-addition to new reinforcement) is also traced out concerning the smaller sphere of load and shows in the sphere of initial loading very big values. Loading of about 600 kg (1323 lb.) causes a very big change of deformation. On the girder built of two part concrete the first part of the deformation of the E-curve-line with its steep branches could not be established.

There were nearly no cracks at the joints.

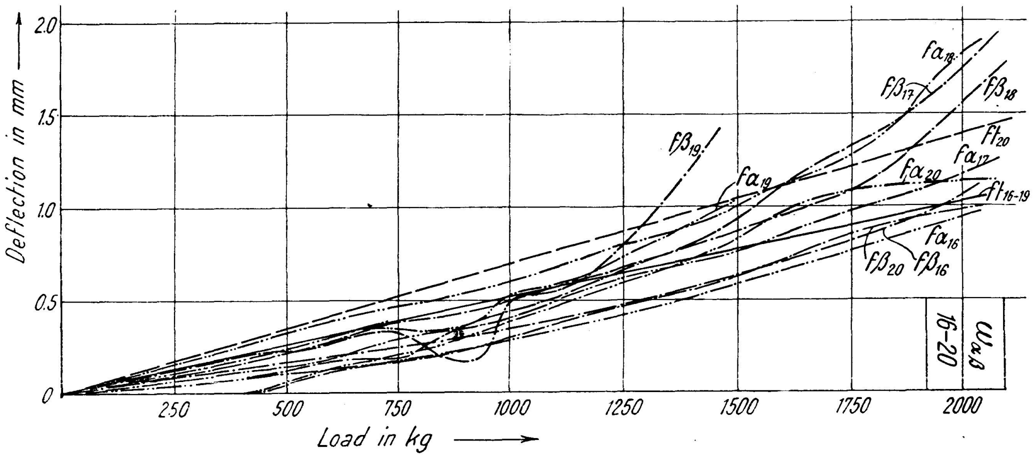


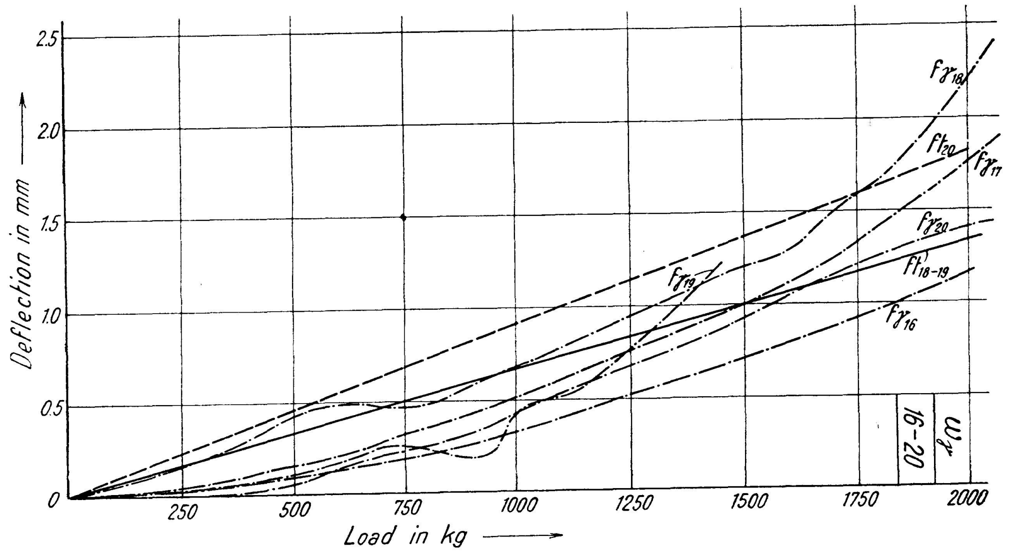


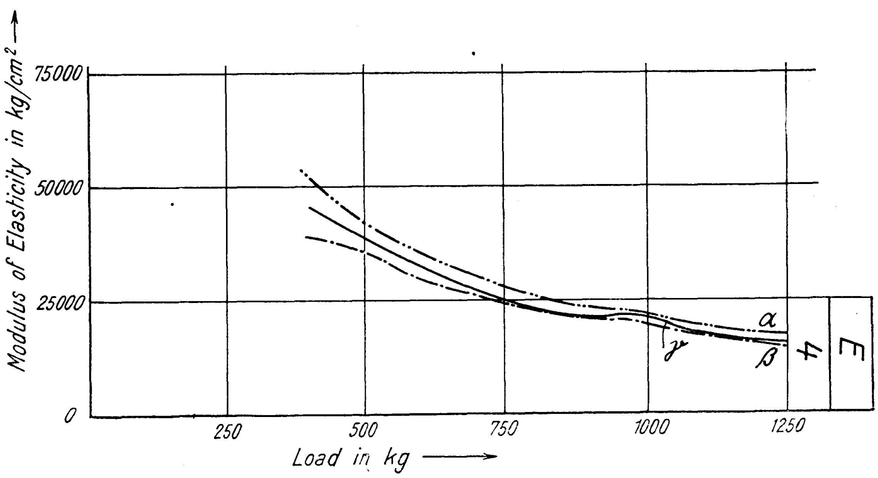
II. Researches concerning break on pillars

For these tests 5 pillars were made, full length 3 m (9,8 feet), transverse-section 15 · 15 cm (5,9 · 5,9 in.), added substance like the girders, quantity of cement 300 kg/m³, reinforcement of 4 – 14 mm. (0,55 in.) steel:

one	pillar	built	homogeneous	with	concrete,	length	3 m (9,8 ft.)
,,	,,	,,	two-part	,,	,,	,,	1 m (3,3 ft.)
,,	,,	,,	,, ,,	,,	,,	,,	1,50 m (4,9 ft.)
,,	,,	,,	,, ,,	,,	,,	,,	2 m (6,6 ft.)
,,	,,	,,	,, ,,	,,	,,	,,	2,25 m (7,4 ft.)







E-curve-lines

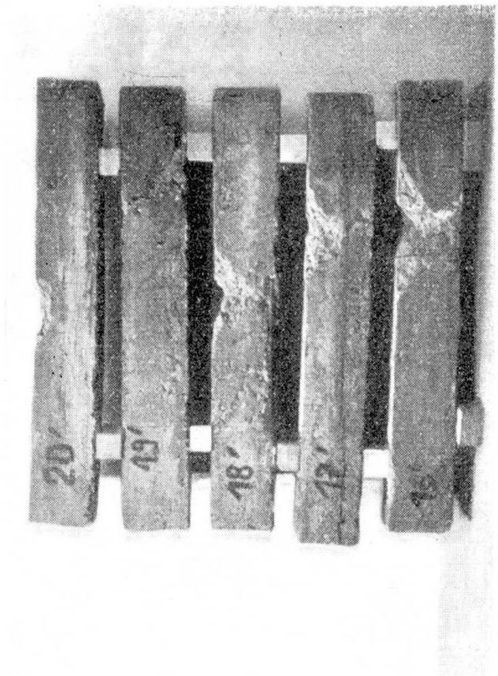


Fig. 5

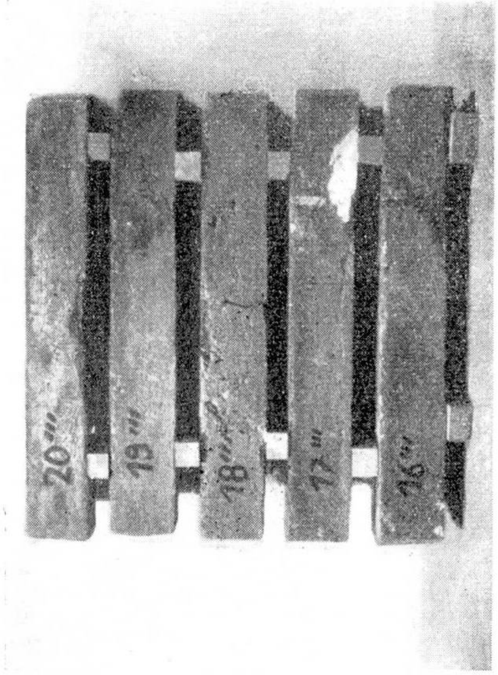


Fig. 7

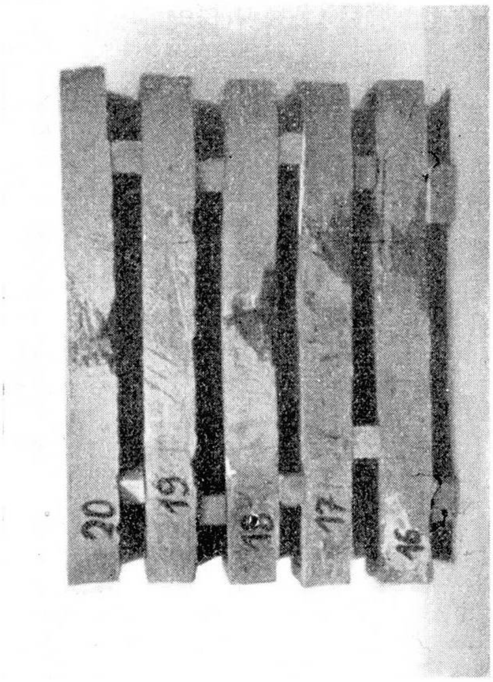


Fig. 4

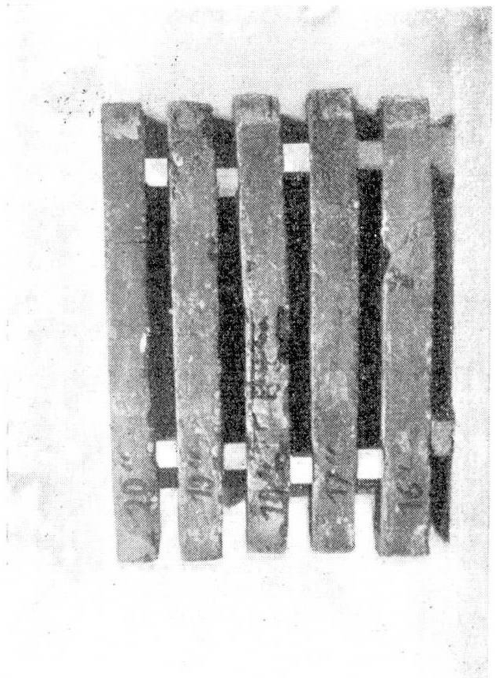
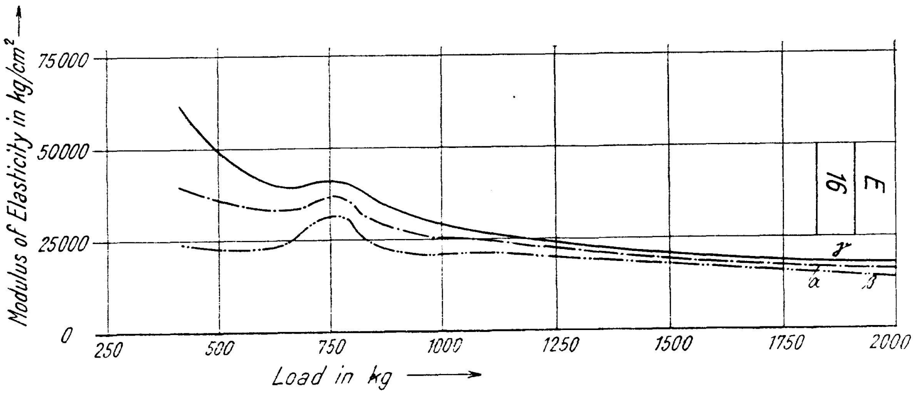
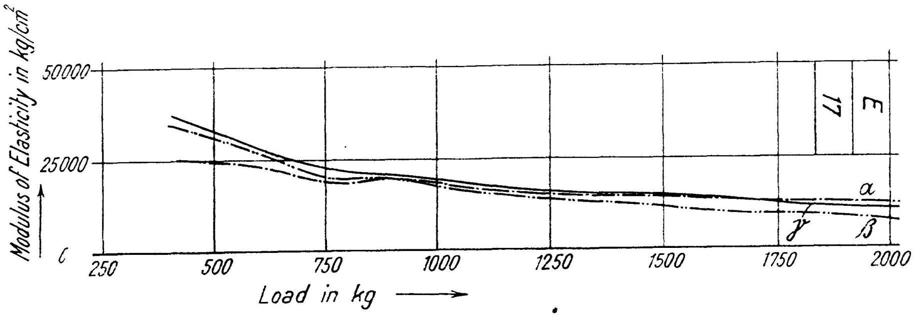


Fig. 6


Modulus of Elasticity in kg/cm²
75000
50000
25000
0
250
500
750
1000
1250
1500
1750
2000
Load in kg
16
E
γ
α
β


Modulus of Elasticity in kg/cm²
50000
25000
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250
500
750
1000
1250
1500
1750
2000
Load in kg
17
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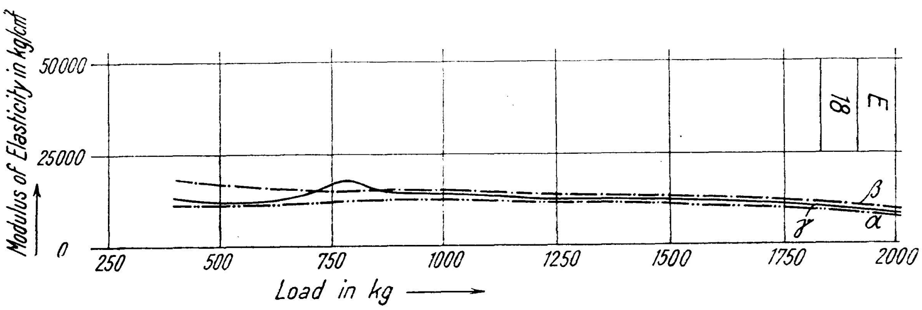
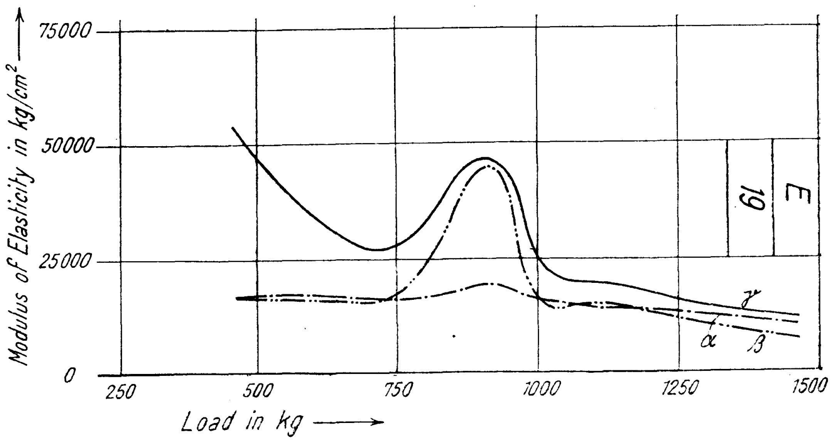

Modulus of Elasticity in kg/cm²
50000
25000
0
250
500
750
1000
1250
1500
1750
2000
Load in kg
18
E
β
γ
α


Modulus of Elasticity in kg/cm²
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Load in kg
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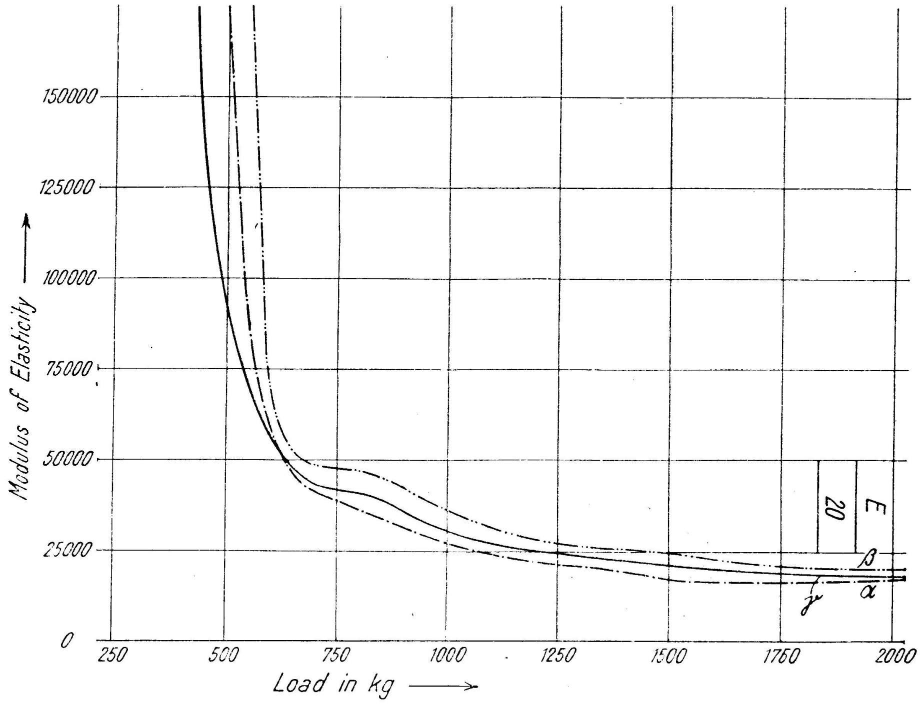

The joining with concrete was made after 6 weeks.

Four weeks after the joining with concrete, the test of breaks were made. During this test no transverse-deflexion was measured. The maximum loading (at failure) was 23 t. The destruction arose without exception at the end of the pillars, i.e. on the older part. Thus the influence on the joining upon the firmness of the construction could not be established.

Summary

The tests made to establish the co-operation between old and newly joined concrete, i.e. tests made concerning deflexion on girders joined with concrete, and the same concerning fracture on pillars joined with concrete, gave the following result:

The girders built with concrete in two stages are, regarding deflexion and bending-stress, not equal to those of homogeneous concrete. They come much sooner into the plastic deformation sphere. Therefore also their carrying capacity is smaller. Girders with steel-reinforcement taken from crashed constructions show great deformation, even at a small loading, also a very small carrying capacity. The fractures both on the girders and the pillars happened nearly without exception beyond the joined areas. Structurally correctly-built joining, even if it seems to be of equal value, can give no guarantee as to the high quality, in a static sense, of the composite construction.

Zusammenfassung

Die Versuche zur Bestimmung des Zusammenwirkens von altem und neu aufgebrachtem Beton, d. h. Deformationsmessungen an Trägern und Bruchversuche an Säulen aus Beton verschiedenen Alters, ergaben folgende Resultate:

Die Träger aus verschiedenaltrigem Beton verhalten sich in Bezug auf Durchbiegung und Biegespannungen nicht gleich wie diejenigen aus homogenem Beton. Sie erreichen die Plastizitätsgrenze viel eher. Darum ist auch ihre Tragkraft kleiner. Träger, deren Armierungsstähle aus zerstörten Konstruktionen gewonnen wurden, zeigen schon große Deformationen bei kleinen Belastungen und haben deswegen eine geringe Tragkraft. Der Bruch erfolgte bei den Trägern und Säulen fast ausnahmslos neben den Verbindungsflächen. Auch konstruktiv richtig durchgeführte Verbindungen, die einer homogenen Konstruktion gleichwertig erscheinen, geben keine Garantie im statischen Sinne, daß dieselbe Qualität erreicht wird.

Résumé

Les recherches effectuées pour établir la coopération de l'ancien et du nouveau béton, comprenant des essais de flexion de poutres re-bétonnées et des essais de rupture de colonnes re-bétonnées, ont donné les resultats suivants:

Les poutres bétonnées en deux temps, du point de vue de la flexion et de la résistance à la flexion, ne sont pas comparables a celles bétonnées en un temps. Elles atteignent le domaine de déformation plastique beaucoup plus vite. Aussi leur solidité est-elle d'autant moindre. Les poutres armées en fers pris à des constructions détruites montrent déjà pour une faible charge une forte déformation, et par conséquant une très petite solidité. Aussi bien les poutres que les colonnes ne montrent qu'exceptionellement des fissures au raccord.

Une jointure, correctement effectuée, même si les deux parties paraissent de même valeur, ne peut offrir aucune garantie, — au sens statique — pour la valeur totale du bâtiment reconstruit.