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CII 2

The use of high-strength steel in ordinary reinforced and prestressed concrete beams

Emploi de l'acier à hautes résistances dans les poutres en béton armé ordinaire et précontraint

Die Verwendung von hochwertigem Stahl in gewöhnlichen und vorgespannten Eisenbetonbalken

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INTRODUCTION

High-strength steel is not generally used in ordinary reinforced concrete because of the danger of excessive cracking with small extension of the concrete. For a long time only mild steel was used and the permissible stresses were limited, but later work-hardened deformed steel bars were introduced and higher steel stresses were allowed, the extent of cracking being limited because of better bond conditions obtained. Plain bars have been excluded mainly because of the very smooth surface. The author had the opportunity of investigating the use of plain high-strength steel bars in connection with spun-concrete poles [1] and ordinary rectangular beams [2] which showed limited width of individual cracks, because owing to good bond a great number of fine cracks developed. Further tests on spun-concrete tubular beams [3] indicated that the concrete tensile zone co-operates greatly up to failure, in spite of the development of cracks. The resistance moment of such slightly reinforced beams was so high that the nominal steel stress, computed for this resistance moment and a lever arm equalling the depth, considerably exceeded the strength of the work-hardened steel used. Extensive tests carried out by Dr. Hajnal-Konyi [4] on beams reinforced with work-hardened square twisted bars in 1942/43 proved that the full strength of such bars could be reached at failure (and not as previously assumed only at the yield-point stress) when the size of the bars was below $\frac{1}{2}$ in. (1·25 cm.). In these tests even a nominal stress in excess of the strength of the steel was obtained. Further tests by Dr. Hajnal-Konyi [5] showed that an increase in ultimate resistance approaching the ultimate strength of cold-worked steel is possible also with bars of larger size provided that an increased bond resistance by surface patterns is ensured.

[1] For references see end of paper

In ordinary reinforced concrete, no use has yet been made of the full strength of high-strength steel. With prestressed concrete, wire of extraordinary high-strength properties has been introduced and there has been no objection to permitting high steel-stresses when, under working load, only compressive stresses occur, as is the case with "full" prestressing, because any possibility of cracking is definitely avoided. The use of such high-strength wire became particularly advantageous when it was realised that losses of prestress, except that due to the creep of the steel, are independent of the steel stress and depend only on the magnitude of the concrete prestress, shrinkage and creep. The ultimate load conditions of prestressed concrete were rarely investigated. However, it has been realised that they are also of importance and must be considered. In the tests [5] also the use, as reinforcement, of thin untensioned wire of very high strength, as preferably used in prestressed concrete, was investigated and it was found that approximately the same ultimate resistance can be attained as when the beam is prestressed. This shows that an investigation of the use of high-strength steel is possible on general lines for ordinary reinforced and prestressed concrete.

CRACKING

It was intended to investigate in the present paper not only the ultimate resistance of work-hardened steel and high-strength wire in concrete beams but also the behaviour of such structures generally. However, in view of the wide field, the question of cracking will be only briefly discussed, while the main part of the paper is devoted to ultimate load conditions.

With regard to cracking, reference may be made to the publications by Professor R. H. Evans,[6] Dr. F. G. Thomas [7] and the author.[8] In reinforced-concrete beams, cracks become visible at a bending moment at which the computed bending tensile-stress in a straight-line distribution for a homogeneous material reaches the so-called modulus of rupture (bending tensile strength). This nominal stress depends on the tensile strength and plasticity of the concrete and on the shape of the cross-section. It may vary between 500 and 1,000 lb./in.^{2} (35 to 70 kg./cm.^{2}) for high-strength concrete. If prestressing is applied, this stress seems to be higher than in ordinary reinforced concrete of the same properties but it would appear that actual cracking commences at the same state, the cracks being invisible at first to the unaided eye. Professor Evans has shown [6] that by measurements with a high-powered microscope fine cracks of a depth of 1/20 in. (1·25 mm.) and a width of 1/15,000 in. (1/600 mm.) may be detected when the unaided eye does not notice cracking. In a recent publication [9] results of investigations by Wenzel and Suhrmann were shown, according to which the intensity of the transmitted pulse was measured by supersonic methods. A reduction of intensity was already noticed at 40% of the load at which cracks became visible, and at the latter load only 40% of the intensity was transmitted. The same investigations also showed that the intensity was reduced to zero long before failure occurred. This method of measurement is based on the fact that even a very narrow air-filled crack reflects the ultrasonic pulse almost completely, and thus a reduction in intensity transmitted indicates the occurrence of cracks. However, when comparing the results mentioned with the actual strength properties, it would appear that such fine and very shallow invisible cracks do not affect the strength properties. With increased prestress the state at which cracks become visible may be further delayed, but commencement of cracking can be inferred from the load–deflection line, even if the cracks are not visible, as was shown in the tests.[10]

From the author's paper [8] it is seen that cracks not exceeding 0·01 in. (0·25 mm.) can be considered as harmless from the point of view of corrosion. With concrete reinforced with ordinary mild steel designed in accordance with the permissible stresses, cracks of even greater width may occur if the concrete is not cured and the influence of shrinkage is great, which is often the case. If the concrete is vibrated a much denser material is obtained and the danger of corrosion is reduced. When considering only the state of cracking, high-strength steel could be used provided the bars were of relatively small cross-section or increased bond resistance were ensured by surface patterns on the reinforcement; even plain high-tensile wire might be used. Nevertheless, it does not seem advisable to use plain high-strength wire in view of the great deflection of such members. By prestressing the entire reinforcement or part of it, cracking under working load can be avoided altogether or its extent limited to a desired degree. For example, it is possible to design a structure in such a way that under ordinary (dead) load no tensile stresses occur and thus any cracks close entirely, while under working load cracks may temporarily open up. As long as this loading is not sustained longer than a certain period, these temporary cracks can be ignored. In view of the author's investigation,[8] even visible fine cracks are harmless with regard to corrosion, but where heavy impact takes place the occurrence of cracks should be avoided altogether unless further investigations have proved that such impact is harmless. In tests [10] it was shown that cracks in prestressed beams with bonded wires close completely on unloading even if the failure load is approached. Advantage can be taken of this great resilience by providing a prestressing force of such magnitude that a considerable range is obtained between noticeable deflection and cracking and failure, as suggested by the author in his paper [11] and embodied in Appendix 2 of the "First Report on Pre-stressed Concrete." [12]

Reference may be made to recent fatigue tests [13] for British Railways carried out at Prof. Campus' Laboratory in Liège on partially prestressed composite members with tensioned and untensioned wires. These members were tested in a cracked state. In one case one million repetitions of loading were applied in a range corresponding to 100 lb./in.^{2} compressive stress and approximately 600 lb./in.^{2} nominal tensile stress (7 and 42 kg./cm.^{2} respectively); after this fatigue test the cracks became entirely invisible. In a second case three million repetitions were applied and after each million the loading was increased so that for the third million nominal tensile stresses of approximately 1,000 lb./in.^{2} (70 kg./cm.^{2}) occurred; after this test very fine cracks were visible. It is noteworthy that just before completion of the third million repetitions, two tensioned wires fractured in gaps provided for affixing the gauges; nevertheless the maximum calculated ultimate resistance was reached at a static failure test, in spite of the previous fatigue loading, as discussed later (see Table VII—slab S2).

Ultimate resistance

The elastic theory is quite suitable for working-load conditions but does not agree with failure conditions. The author showed in 1935/7 [1, 14] that with ordinary reinforced concrete for various percentages of reinforcement quite different factors of safety are obtained when the design is based on permissible stresses. If cases are excluded at which failure occurs owing to shear or slipping of the steel, two cases must be distinguished, i.e. under-reinforced beams when failure is primarily due to the steel (either fracture or excessive elongation of steel followed by crushing of the concrete), and over-reinforced beams where failure is due to crushing of the concrete at a state when extension of the steel is relatively small and no warning is given of imminent failure. Some of the special methods which were suggested a long time ago have been discussed by Prof. R. H. Evans,[15] Dr. K. Hajnal-Konyi [16] and the author.[1, 14, 17] The following names and dates may be mentioned: L. J. Mensch (1914), H. Kempton-Dyson (1922), F. Emperger (1931), Prof. F. Stüssi (1932), Dr. F. Gebauer (1933), Dr. C. Schreyer (1933), S. Steuermann (1933), Dr. E. Bittner (1935/6), Prof. R. Saliger (1936), Charles S. Whitney (1937), Kenneth C. Cox (1941), Prof. V. P. Jensen (1943) and R. H. Squire (1943). In addition to these methods three further suggestions may be mentioned, e.g. those of Prof. A. L. L. Baker,[18] Mr. J. W. King [19] and Prof. Hjalmar Granholm.[20]

Professor R. H. Evans has shown in his paper [15] that there is little difference in the results of the various methods, and it seems therefore most advisable to employ the simplest solution. All methods are only approximations, though it is claimed by some proposers that they have presented exact formulae based on strain consideration. However, it must not be forgotten that there is a great variety in the behaviour of concretes of different mixes, and practically any property may be obtained.[14] Whitney has shown [21] that the resistance stress at failure $Q=M_m/(b\,.\,d^2)$ approximates to $f'_c/3$, where f'_c is the cylinder strength if f'_c exceeds 2,500 lb./in.,[2] while for lower values of f'_c higher values apply for Q/f'_c. Whitney suggested a rectangular compressive distribution of a stress $0{\cdot}85\,f'_c$ balancing the ultimate steel resistance. Kenneth C. Cox [22] has modified this formula by introducing the entire cylinder strength instead of $0{\cdot}85\,f'_c$ for the rectangular stress distribution; but Whitney stated in the discussion that this applied only if the cylinder strength was obtained from specimens differing in size from the standard cylinder. Prof. Evans, in his paper,[15] has come to the conclusion that the prism strength with high strength values approaches the cube strength and has introduced the cube strength for the compressive stress. Fig. 1 shows the results of Whitney's and Evans' investigations. The author, who took part in the discussion on these papers,[21, 22] has used stress distributions according to Cox in his publications [23, 24, 25] when dealing with ultimate load conditions, but would like to modify this method slightly in the following paragraphs.

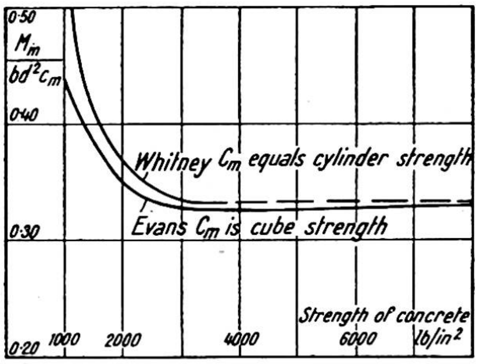
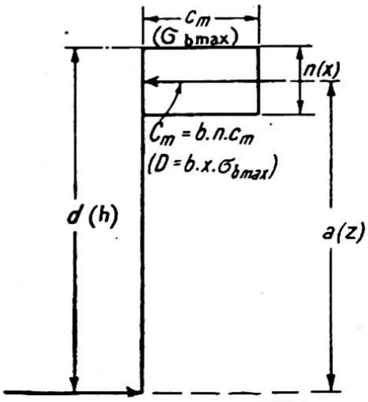


Fig. 1

The magnitude of the maximum compressive stress c_m, as shown in fig. 2(*a*),* depends mainly on the strength and plasticity of the concrete used. It will be appreciated that this measure could only be considered as a strength value if this strength were obtained from specimens of definite size. Take, for example, prisms; quite different sizes are being used with the consequence of different strength values. Thus the stress c_m cannot be taken as a strength value; but as a stress it may be considered as dependent on the strength; e.g. it can be assumed that $c_m=G\,.\,c_u$, where c_u is the cube strength and G is a coefficient generally varying between 0·6 and 0·8, but c_m may also in certain circumstances equal the prism strength c_p. The second modification of the Whitney method consists in the assumption that the maximum equivalent depth of the rectangular stress-distribution is half the depth d, resulting in $M_{max}=0{\cdot}375\ bd^2c_m$. If G is taken as 0·6 a value of $M_{max}=0{\cdot}225\ bd^2c_u$ is

* In fig. 2, the formulae written with the symbols used in German-speaking countries are shown in parentheses.

obtained, as suggested by the author [25] and introduced in the "First Report on Prestressed Concrete." [12] Preliminary investigations have proved that this assumption agrees very well with test results, as may be seen from the charts, figs. 6 and 7.



$T_m = A_t t_u \qquad p = \frac{100 A_t}{bd}$

$(Z = F_e \mathfrak{S}_{e\max}) \qquad \left(\mu = \frac{100 F_e}{bh}\right)$

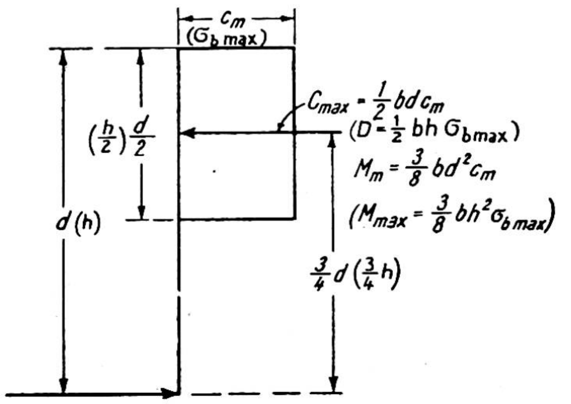
$M_m = A_t t_u\left(d - \frac{A_t t_u}{2bc_m}\right)$

$R = \frac{Q}{c_m} = \frac{M_m}{bd^2 c_m} = 2vp(1-vp)$

where $Q = \frac{M_m}{bd^2}$ and $v = \frac{t_u}{200c_m}$

$J = \frac{100 M_m}{bd^2 t_u p(1-vp)} = 1$

Fig. 2(*a*)



$T_u = A_t t_u$

$(Z = F_e \mathfrak{S}_{e\max})$

$\left[\frac{M_{\max}}{bh^2 \mathfrak{S}_{e\max}} = 2v\mu(1-v\mu) \quad Q = \frac{M_{\max}}{bh^2} \text{ and } v = \frac{\mathfrak{S}_{e\max}}{200 \mathfrak{S}_{b\max}}\right]$

$\left[J = \frac{100 M_{\max}}{bh^2 \mathfrak{S}_{e\max}\mu(1-v\mu)} = 1\right]$

Fig. 2(*b*)

Fig. 2 shows the stress distribution at failure for (*a*) an under-reinforced section and (*b*) for balanced design, including formulae for balancing the ultimate resistances. The percentage of reinforcement for balanced design $\bar{p}$ is determined by the relation $\bar{p} = 0{\cdot}25/v = 50c_m/t_u$, where $v = t_u/200c_m$; t_u is the ultimate steel-strength (in certain cases it should be replaced by t_y, the yield-point stress), and c_m is the maximum concrete stress. The stress $Q = M_m/(bd^2)$ due to maximum bending moment M_m is investigated with regard to c_m and t_u. The stress-ratio $R = Q/c_m$, obtained from the test results, can be compared with the value $2vp(1-vp)$ obtained from the force equilibrium for under-reinforced beams. If R is greater than $2vp(1-vp)$, the calculation gives safe values. For over-reinforced beams ($p \geqslant \bar{p}$) the theoretical value is 0·375. Similarly the utilisation of the steel can be investigated by computing $J = 100\, Q/(t_u \,.\, p(1-vp))$; the theoretical value is unity for under-reinforced sections and $\bar{p}/p$ for over-reinforced sections. The steel strength is fully utilised if $J \geqslant 1$ or $J \geqslant \bar{p}/p$, where $p \geqslant \bar{p}$, while lower values of J indicate that full use is not made of the steel strength. J represents, in fact, the ratio t_m/t_u, were t_m is the steel stress in a cracked section calculated for the maximum bending moment. When the values for J obtained from test results are greatly in excess of the theoretical values, it must be assumed that the concrete tensile zone co-operates in spite of its interruption by cracks. In such a case the stress distributions according to fig. 2 would have to be modified by considering an average concrete tensile resistance, as, for example, has been suggested by the author in the discussion to Cox's paper.[22]

These formulae have been investigated for a number of tests on prestressed and ordinary reinforced concrete beams, the cross-sections of which are shown in figs. 3 and 4 respectively. Fig. 3(*a*) relates to unpublished tests carried out by Stott at the

University of Leeds and fig. 3(*b*) to similar tests carried out by Revesz at the Imperial College, London. The author has obtained the data shown in Tables I and II from Profs. R. H. Evans and A. L. L. Baker respectively, to whom as well as to Messrs. Stott and Revesz he expresses his thanks. The test results are to be discussed in more detail in theses.

The results of published Swiss tests,[27] omitting beam II reinforced with mild steel, and those of the Brixton School of Building [28, 29] have been investigated, cross-sections

TABLE I

Data obtained from Prof. R. H. Evans regarding tests carried out by Mr. J. P. Stott at the University of Leeds. (Cross-section, see fig. 3(*a*); span 10 ft. for beams 1–17 and 3 ft. 4 in. for beam 18; loading at third points.)

Mark	b	d	c_u*	t_u	Wires† top	Wires† bottom	Failure‡ moment	$R=\dfrac{M_m}{bd^2c_u}$ §
	in.		lb./in.²	tons/in.²	number		in.-tons	
1	2·5	8·13	7,000	140	11	45	136·0	0·264
2	2·5	8·21	7,430		9	36	174·6	0·321
3	2·5	8·13			8	31	152·6	0·285
4	2·63	8·13			6	27	147·2	0·271
5	2·56	8·13			8	32	146·8	0·269
6	2·56	8·23	9,100	120	8	31	126·4	0·189
7	2·56	8·12			8	29	123·4	0·185
8	2·53	8·0			7	27	119·4	0·179
9	2·59	8·1			7	25	107·4	0·161
10	2·50	8·25	8,260		9	35	152·0	0·250
11	2·60	8·26			9	33	149·8	0·247
12	2·56	8·20			10	39	164·4	0·271
13	2·56	8·19			10	37	152·4	0·251
14	2·50	8·13			11	44	180·4	0·297
15	2·56	8·06			11	42	154·8	0·255
16	2·50	8·06			10	41	164·4	0·271
17	2·50	8·06			10	39	164·6	0·272
18	2·50	8·06			10	39	152·0	0·254

* Concrete strength on 4-in. cubes.

† All wires were of 0·08 in. diameter; initial prestress for the individual beams varying between 76·6 and 81·7 tons/in.²

‡ The bending moment, due to dead load, of 1·7 in.-tons must be added to these values.

§ These values include the bending moment due to dead load of beam and the influence of the residual tension in top steel at failure.

TABLE II

Data obtained from Prof. A. L. L. Baker regarding tests carried out by Mr. S. Revesz at the Imperial College of Science in London. (Cross-section, see fig. 3(*b*); span 14 ft.; loading at third points.)

Mark	Group	b*	d	d'	c_u†	Number of Wires‡	p_i ‖	Failure¶ moment	At failure	
		in.			lb./in.^{2}		tons/in.^{2}	in.-tons	$\frac{\text{Strain } A'_t}{\text{Strain } A_t}$	$\frac{\text{Stress } A'_t}{\text{Stress } A_t}$
K	Ia Fig. 3(*b*)i	3·00	5·03	0·55	7,840	8	83·58 to 84·44	38·56	—	—
A		2·92	4·61		7,820	12		49·42		
B		2·91	4·85		7,820			55·3		
E		2·88	5·00		7,650		0	51·38		
H		2·93	4·77		7,840		52·4	57·82		
D	Ib Fig. 3(*b*)ii	4·25	6·92	3·16	4,010	12	84·44	74·40	—	—
C			6·72		3,090	12	84·44	92·04		
			6·05			2§	45·8			
F	II Fig. 3(*b*)iii	21·5	7·74	4·00	2,280	12	0	107·88	0·407	0·567
M			6·96	2·56	2,240		92	93·32	0·195	0·702
L			7·46	3·35	6,560			107·88	0·42	0·81
G			7·01	2·62	5,560		52·4	99·48	0·304	0·637
J			7·78	3·25	3,720	8	84·4	69·14	0·29	0·76

* The width b in Group Ia is obtained if the co-operation of the top reinforcement A'_t is taken into account, based on strain measurements.

† Concrete strength on 6-in. cubes.

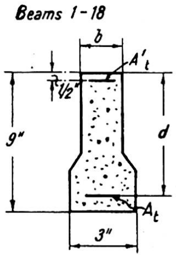
‡ The number of wires relates to the bottom reinforcement. In each beam two top wires were provided. The wire is throughout 12-gauge (area per wire 0·0087 in.^{2}) of a strength of 132 tons/in.^{2} and bonded except for beam C (see next note).

§ In addition to twelve bonded wires, as specified above, two non-bonded wires 0·2 in. diameter of a strength of 107 tons/in.^{2} were provided.

‖ The initial prestress in the two top wires was equal to that in the bottom wires. The prestress was transferred when the concrete strength was approximately 5,500 lb./in.^{2}

¶ Failure in all cases occurred owing to fracture of tensile wire, except for beams C and E (crushing of concrete) and J (horizontal shear).

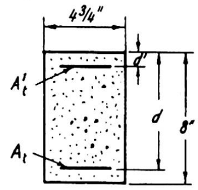
being shown in figs. 3(*c*) and 3(*d*) respectively. Furthermore, two types of slabs according to fig. 3(*e*) are included as tested for British Railways.[13] Particulars of the slabs S I and S II have not yet been printed but were given by the author in a lecture.[13] These tests relate to 9-ft. long members loaded as cantilevers at both sides 9 in. away from the ends and supported at two points each 9 in. from the centre. The tensioned reinforcement consisted of eight wires 0·2 in. diameter placed in groups of four in grooves which were later filled with cement mortar, Magnel-Blaton anchorages being provided at the ends. Each slab S I and S II contained four untensioned wires in the compression zones and four additional untensioned wires were provided in the tensile zone of S II. It may be pointed out that S 2 also contained untensioned wires, as proposed by the author when suggesting partial prestressing.[23, 30]



b varying between 2·5" & 2·63"
d " " 8" & 8·26"

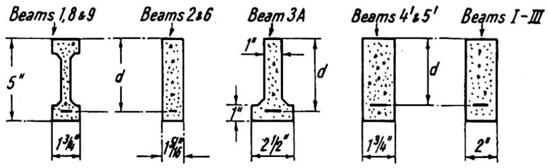
• TESTS LEEDS 1950 +

Fig. 3(a)



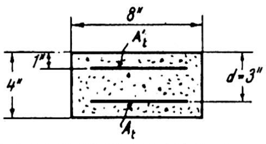
SWISS TESTS

Fig. 3(c)

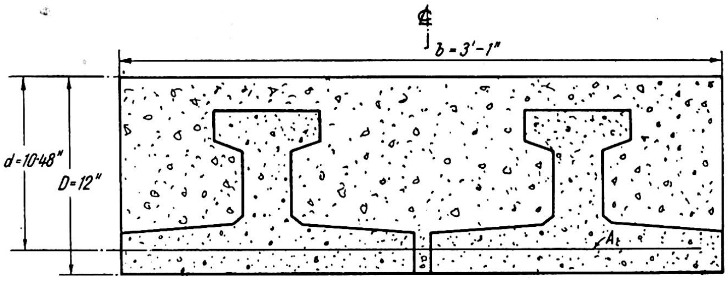


TESTS BRIXTON 1950/51 ×

Fig. 3(d)



TEST BRITISH RAILWAYS
EASTERN REGION



S2

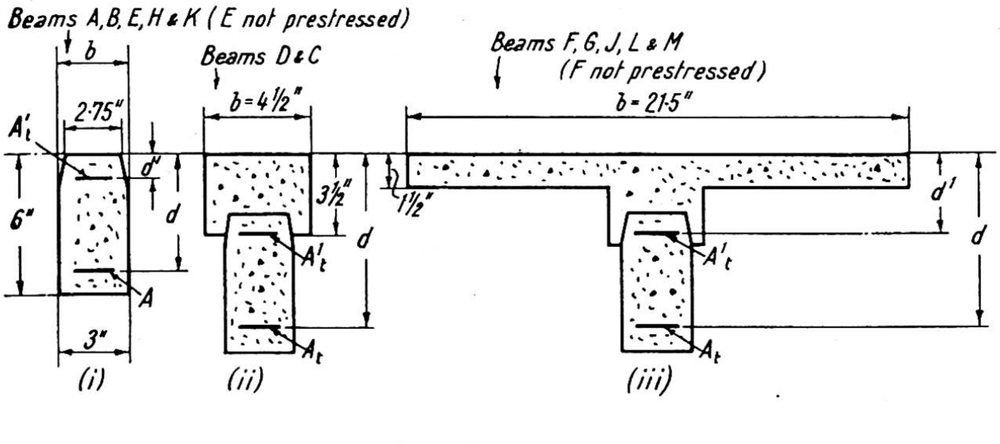
FATIGUE TEST - RAILWAY EXECUTIVE

Fig. 3(e)

TABLE III
Tests of Stott, Leeds (see Table I)

Mark	p %	Q lb./in.^{2}	$v=t_u/200c_m$*	$p/\bar{p}$	R	$2vp(1-vp)$	J
1	1·11	1,850		1·42	0·377	0·428	0·775
2	0·875	2,320		1·064	0·459	0·340	1·17
3	0·762	2,070	0·304	1·028	0·407	0·356	1·13
4	0·630	1,900		0·764	0·387	0·309	1·18
5	0·768	1,940		0·956	0·384	0·359	1·05
6	0·735	1,640		0·620	0·270	0·261	0·985
7	0·697	1,645	0·202	0·592	0·264	0·252	1·03
8	0·667	1,658		0·560	0·255	0·240	1·085
9	0·595	1,420		0·504	0·230	0·220	1·01
10	0·85	2,016		0·79	0·356	0·317	1·22
11	0·77	1,900		0·71	0·352	0·294	1·12
12	0·93	2,150		0·86	0·387	0·338	1·27
13	0·98	2,000		0·82	0·359	0·326	1·06
14	1·08	2,470	0·232	1·0	0·425	0·375	1·125
15	1·02	2,090		0·944	0·364	0·360	1·01
16	1·02	2,280		0·944	0·387	0·360	1·11
17	0·97	2,280		0·900	0·385	0·348	1·25
18	0·97	2,100		0·900	0·363	0·348	1·15

* $c_m=0·7\ c_u$.



TESTS IMPERIAL COLLEGE ⊕
1949/50

Fig. 3(*b*)

TABLE IV

Tests of Revesz, Imperial College (see Table II)

Mark	p %	Q lb./in.^{2}	v†	$p/\bar{p}$	R	$2vp(1-vp)$	J
K	0·461	1,140	0·314	0·579	0·243	0·248	0·978
A	0·776	1,780		0·976	0·378	0·369	1·025
B	0·740	1,810		0·932	0·384	0·355	1·084
E*	0·724	1,595	0·322	0·932	0·348	0·357	0·976
H	0·747	1,940	0·314	0·940	0·356	0·359	1·145
D	0·355	818	0·613	0·87	0·339	0·341	0·996
C	0·605	1,160	0·740	1·764	0·614	0·493	1·25
F*	0·0714	208	1·078	0·288	0·148	0·136	1·04
M	0·0835	229	1·097	0·366	0·171	0·167	1·02
L	0·079	231	0·375	0·118	0·059	0·057	1·02
G	0·0815	239	0·443	0·145	0·072	0·070	1·03
J	0·0507	143	0·660	0·133	0·065	0·064	1·01

* Non prestressed. † $c_m = 0{\cdot}6\, c_u$.

TABLE V

Swiss tests (Section, see fig. 3(c))

Mark	p %	Q lb./in.^{2}	v	$p/\bar{p}$	R	$2vp(1-vp)$	J	c_m lb./in.^{2}	t^u lb./in.^{2}
I	0·5	1,175	0·233	0·446	0·235	0·206	1·14	5,000	233,000
III		1,050			0·210	0·200	1·02		
VII		995	0·256	0·518	0·221	0·215	1·04	4,500	
IV	0·394	850	0·219	0·346	0·170	0·158	1·08	5,000	219,000
V		800			0·160	0·158	1·02		
VI		605	0·274	0·432	0·151	0·193	0·785	4,000	
VIII	0·222	618	0·275	0·244	0·124	0·114	1·08	5,000	275,000
IX	0·0975	296	0·306	0·119	0·066	0·052	1·14	4,500	

TABLE VI

Tests at Brixton School of Building (Section, see fig. 3(d), t_u=138 tons/in.^{2})

Mark	p %	Q lb./in.^{2}	v	$p/\bar{p}$	R	$2vp(1-vp)$	J	c_m lb./in.^{2}
1	0·22	641	0·386	0·340	0·160	0·155	1·03	4,000
2	0·29	402		0·448	0·200	0·199	1·01	
3A	1·58	885		2·40	0·495	0·480	1·03	
4′	0·60	1,432		0·928	0·358	0·356	1·01	
4A	0·47	1,212		0·726	0·303	0·297	1·02	
6	0·27	786		0·418	0·201	0·187	1·05	
8*	0·22	461		0·34	0·114	0·155	0·735	
9		625			0·156		1·00	
I*	0·224	433	0·341	0·306	0·096	0·141	0·68	$\frac{2}{3}\times 6{,}800$
II		765			0·169		1·23	
III	0·50	1,390	0·288	0·576	0·256	0·247	1·05	$\frac{2}{3}\times 8{,}040$

* Wires non-bonded.

TABLE VII

Tests of British Railways (Section S2, see fig. 3(e); Section S I and S II, see fig. 3(e))

Mark	p %	Q lb./in.^{2}	v	$p/\bar{p}$	R	$2vp(1-vp)$	J	c_m lb./in.^{2}	t_u lb./in.^{2}	Notes
S2	0·483	942	1·735	0·334	0·153	0·153	1·005	6,150	213,000	After fatigue loading
SI	1·05	1,866	0·224	0·940	0·373	0·359	1·037	5,000	224,000	Eight tensioned wires 0·2 in. dia.
SII	1·57	2,653		1·412	0·532	0·360	1·03			Eight tensioned plus four untensioned wires 0·2 in. dia.

TABLE VIII

Tests on spun-concrete tubes (Section, fig. 4(a), t_u=65,000 lb./in.², c_m=8,000 lb./in.²)

Mark	p	Q	v	$p/\bar{p}$	R	$2vp(1-vp)$	J	Notes
	%	lb./in.²						
9b	0·38	445		0·06	0·056	0·030	1·83	
7a	1·26	910		0·20	0·114	0·097	1·175	Tubular Beams
10b	1·92	1,280	0·0406	0·31	0·160	0·144	1·115	
13b	0·61	620		0·10	0·078	0·049	1·605	Inverted tubular T. beams
14a	2·15	1,550		0·35	0·194	0·160	1·215	

TABLE IX

Tests on rectangular beams, Vienna (Section, fig. 4(b))

Mark	p	Q	v	$p/\bar{p}$	R	$2vp(1-vp)$	J	t_y	c_u*
	%	lb./in.²						lb./in.²	
22	0·38	502	0·0850	0·128	0·0895	0·0619	1·44	95,000	
23	0·84	911	0·0775	0·26	0·163	0·122	1·34	87,000	8,400
24	1·47	1,375	0·0811	0·476	0·246	0·210	1·17	91,000	
4	0·39	378	0·346	0·54	0·280	0·234	1·175	95,000	2,060
5	0·84	480	0·316	1·056	0·373	0·389	0·85	87,000	

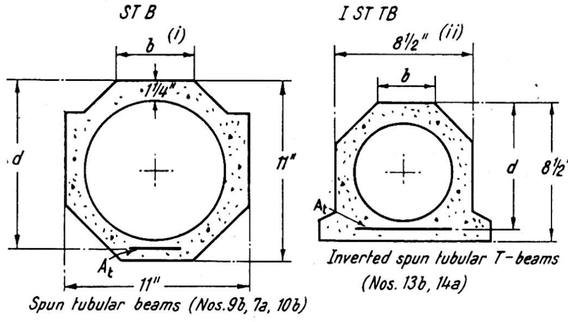
* $c_m=\frac{2}{3}\,c_u$.

TABLE X

Tests of Dr. Hajnal-Konyi, 1942 (Section fig. 4(c))

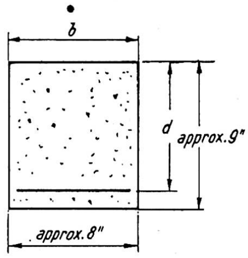
Mark	p	Q	v	$p/\bar{p}$	R	$2vp(1-vp)$	J	c_p	t_u	Notes
	%	lb./in.²						lb./in.²		
20	0·214	193·8	0·208	0·174	0·108	0·083	1·25	1,800	75,000	Twisted bar, 5-gauge
25	1·19	647	0·184	0·832	0·323	0·327	0·945*	2,000	73,600	Twisted bar, ½-in.
27	1·19	811	0·143	0·680	0·324	0·282	1·21	2,500	71,500	
33	0·353	310		0·282	0·155	0·131	1·18			
34	0·562	482	0·20	0·450	0·241	0·199	1·23	2,000	80,000	Twisted bar, 5-gauge
31	0·750	594		0·600	0·297	0·255	1·16			
32	0·938	725		0·670	0·362	0·304	1·19			

* below 1·0, since bars under-twisted



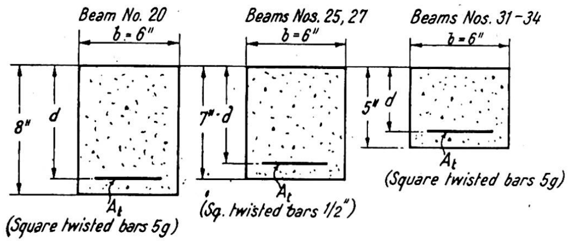
TESTS ON SPUN CONCRETE SPECIMENS REINFORCED WITH I STEG STEEL

Fig. 4(*a*)



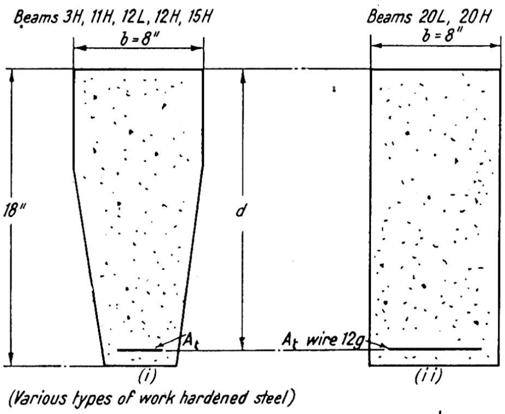
TESTS ON RECTANGULAR BEAMS REINFORCED WITH HIGH STRENGTH STEEL (Nos. 4, 5, 22, 23 & 24)

Fig. 4(*b*)



TESTS DR. HAJNAL-KONYI 1943

Fig. 4(*c*)



TESTS DR. HAJNAL-KONYI 1951

Fig. 4(*d*)

Fig. 4 relates to ordinary reinforced concrete; fig. 4(*a*) refers to the author's tests on spun-concrete tubular beams,[3] fig. 4(*b*) to the author's tests on rectangular beams,[2] and figs. 4(*c*) and 4(*d*) to Dr. Hajnal-Konyi's tests [4, 5] respectively.

TABLE XI

Tests of Dr. Hajnal-Konyi, 1951 (Section, fig. 4(*d*))

Mark	p %	Q lb./in.²	v	$p/\bar{p}$	R	$2vp(1-vp)$	J	c_u lb./in.²	t_u lb./in.²	t_y lb./in.²	Notes
3H	0·608	396	0·0873	0·213	0·096	0·100	0·95	6,200	72,130	—	Square twisted bar, $\frac{7}{8}$ in. dia.
			0·0760	0·185		0·088	1·09		—	62,940	
11H	0·584	447	0·1043	0·244	0·111	0·115	0·97	6,050	84,220	—	Tor steel 1 in. dia.
			0·0875	0·205		0·097	1·11		—	70,780	
12L	0·505	417	0·1736	0·393	0·170	0·177	0·97	3,230	85,300	—	Indented twisted bar 1 in.
			0·1490	0·337		0·154	1·10		—	73,200	
12H	0·570	461	0·1168	0·266	0·121	0·124	1·015	5,760	86,000	—	
			0·0997	0·227		0·107	1·165		—	73,700	
15H	0·580	456	0·1113	0·258	0·121	0·121	1·00	5,890	83,600	—	American ribbed type, twisted 1 in.
			0·0935	0·217		0·103	1·19		—	70,300	
20L	0·178	456	0·478*	0·340	0·163	0·155	1·05	3,990*	268,000	—	High-tensile wire
20H	0·180	491	0·355*	0·256	0·130	0·120	1·09	5,400*			

* $c_m = 0{\cdot}7\ c_u$.

The investigation of these test results with regard to the presented formulae is shown in Tables III–XI. It is obviously of greatest importance to select the right value c_m when calculating v, assuming that t_u is accurately known, which will be the case generally. In the tests in fig. 3(*a*) a ratio $G = c_m/c_u = 0{\cdot}7$ has been taken into account, while with regard to the tests in fig. 3(*b*) $G = 0{\cdot}6$ is still rather on the low side. If the wire fractures in a certain case, it must be expected that J is not less than unity. Hence for $J = 1$, the corresponding value v and thus c_m can be computed, resulting in $c_m = \dfrac{(t_u p)^2}{200\,(t_u p - 100\,Q)}$ which results for beam K, in $c_m = 3{,}980$ lb./in.² which would have corresponded to a ratio $G = 0{\cdot}515$ instead of 0·6 as taken into account. In the case of the Swiss tests, the concrete strength was not known accurately, but from the fact that in specimen III the wires fractured and the steel strength was given, it was possible to assume c_m. In the Brixton and British Railways tests the published prism strength values have been taken into account. For the spun-concrete tubular beams $c_m = 8{,}000$ lb./in.² has been assumed in view of the extraordinary strength properties of these specimens and the ultimate strength of the Isteg steel has been considered, although there was a distinct yield point of this reinforcement. Nevertheless an extraordinary excess over unity was obtained for the ratio J. The high-strength steel used in the rectangular beams tested by the author had a distinct yield point, and this stress and two-thirds of the concrete cube strength were taken in analysing the results. In Dr. Hajnal-Konyi's tests the published prism strength values have been used, except for 20L and 20H, when 70% of the cube strength as with the Stott tests has been used. For the specimens (fig. 4(d)i) the ultimate strength and the yield point stress have been investigated. The examples (fig. 4(d)ii) were not included in the paper [5] but were given during its presentation and published in *Magazine of Concrete Research* in March, 1952.

The results evaluated in the Tables III–XI have been plotted in charts figs. 5–7.

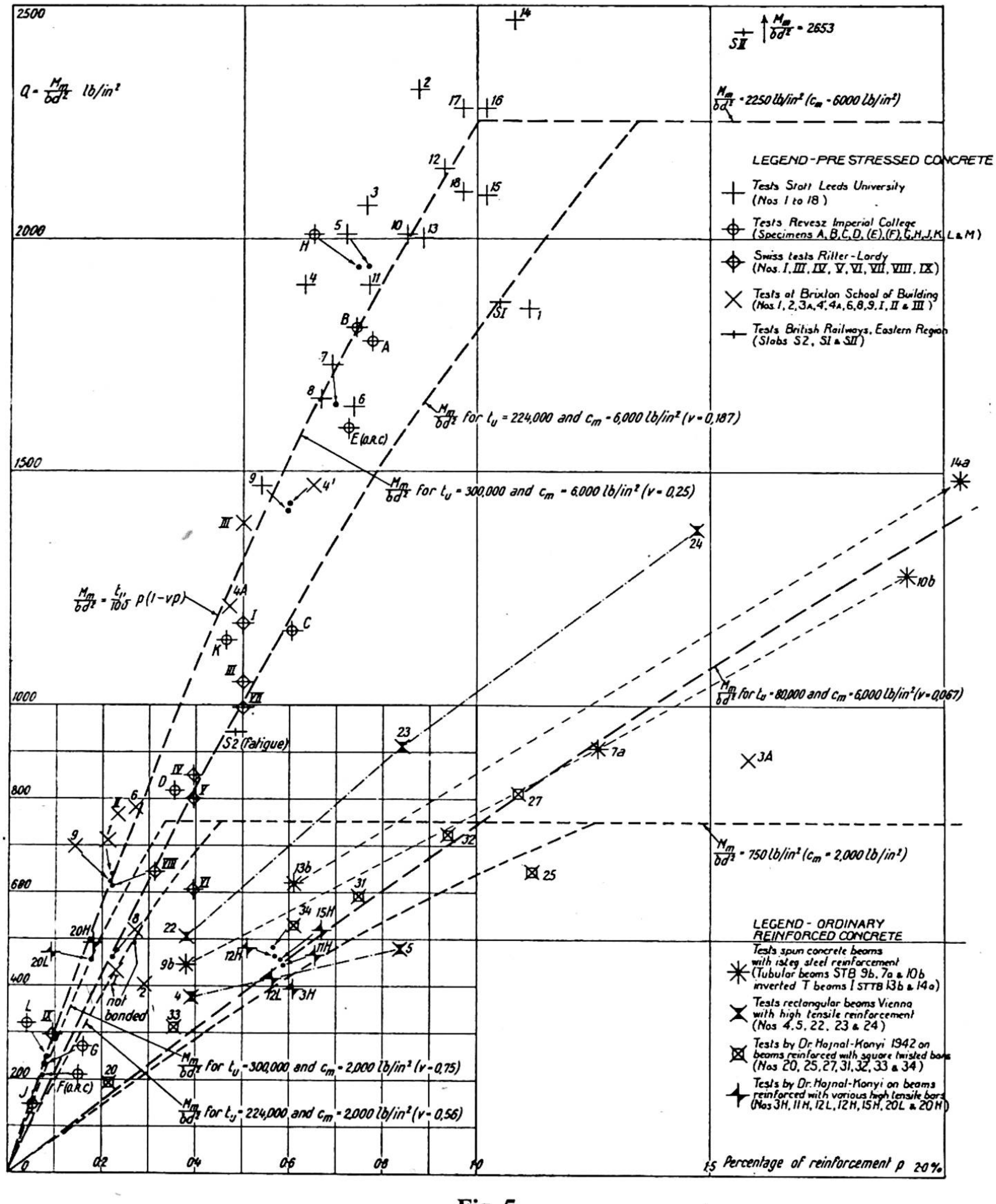


Fig. 5

Fig. 5 shows the relation between the percentage p and the stress $Q = M_m/(bd^2)$. In addition to test results theoretical values have been plotted for $t_u = 80{,}000$, and 224,000 and 300,000 lb./in.², and in each case two concrete values have been distinguished: $c_m = 2{,}000$ and 6,000 lb./in.² According to fig. 1 the value for balanced design for a low stress, such as $c_m = 2{,}000$ lb./in.² would be rather higher than $c_m/3$. In figs. 6 and 7 prestressed and ordinary reinforced concrete are separated. Fig. 6 indicates how far the concrete strength is utilised, while fig. 7 shows the exploitation of the steel strength in exaggerated presentation, a difference of 5 % appearing as a great deviation. The chart fig. 6 has been plotted in such a way that the abscissa represents $2\nu p(1-\nu p)$, while the ordinate is R; thus a straight line between the origin and $R = 2\nu p(1-\nu p) = 0{\cdot}375$ is obtained. In fig. 7 the values of J are plotted against $p/\bar{p}$. A very good agreement between the minimum values indicated in figs. 6 and 7 and the actual

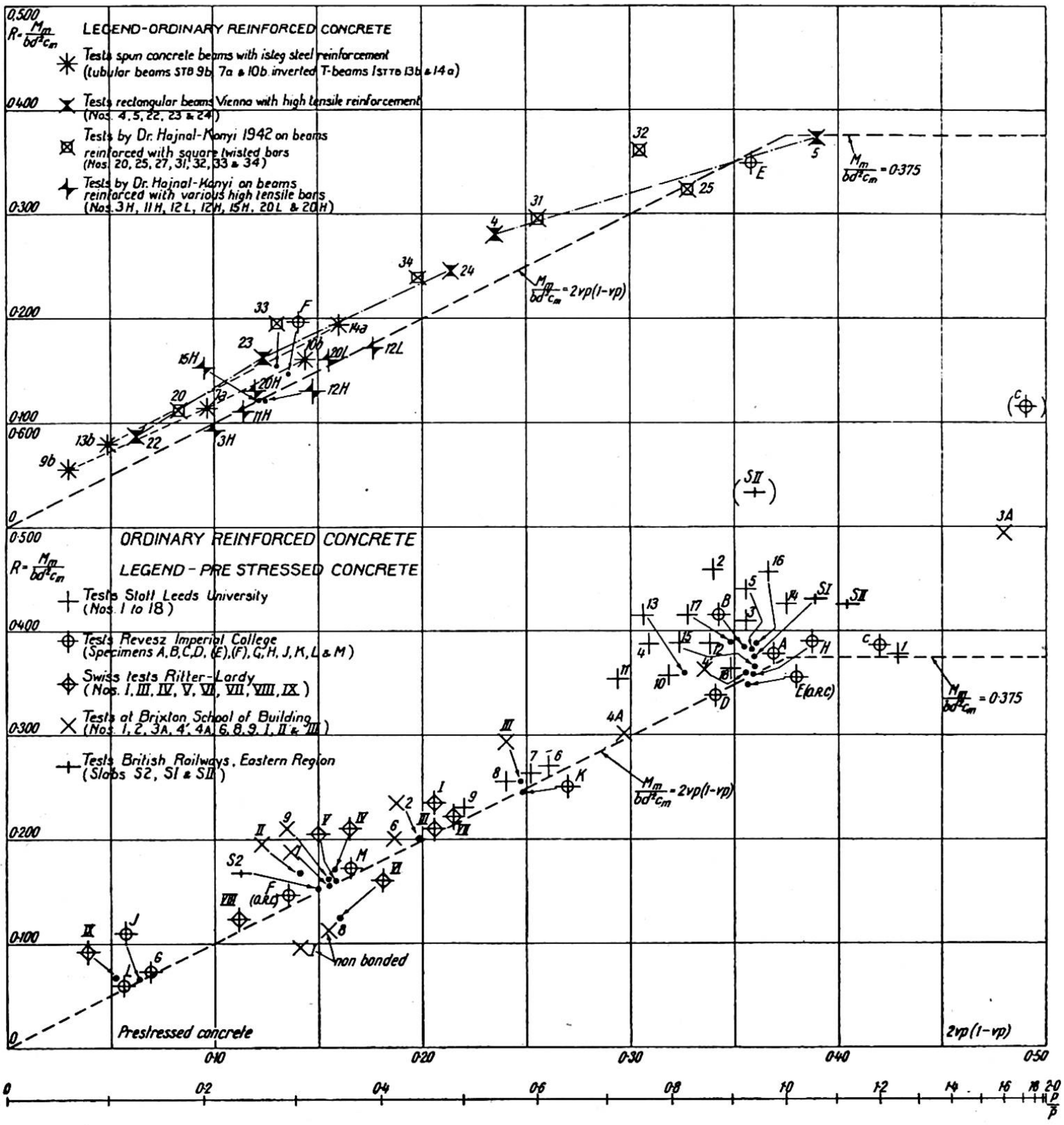


Fig. 6 (Note:—the point 27 is not shown)

values is seen, and it can be concluded that safe values are obtained if G is assumed as 0·6.

However, there are a few points which require further discussion. The slab S II of the British Railways and the beam C of the Imperial College show very high values of R, much exceeding the expected limit of 0·375. In the first case the reason is that the compressive reinforcement consisting of four untensioned high-strength wires has not been taken into account in Table VII. However, when considering a nominal width of 10 in., corrected values are obtained as plotted in figs. 6 and 7 in addition to the original values, indicated by brackets. The values for beam C have been obtained from $c_m = 0{\cdot}6 \times 3{,}090 = 1{,}854$ lb./in.^{2}, a very low value, resulting in $R = 0{\cdot}614$, which seems to be an impossible value and must be excluded. It must be assumed that

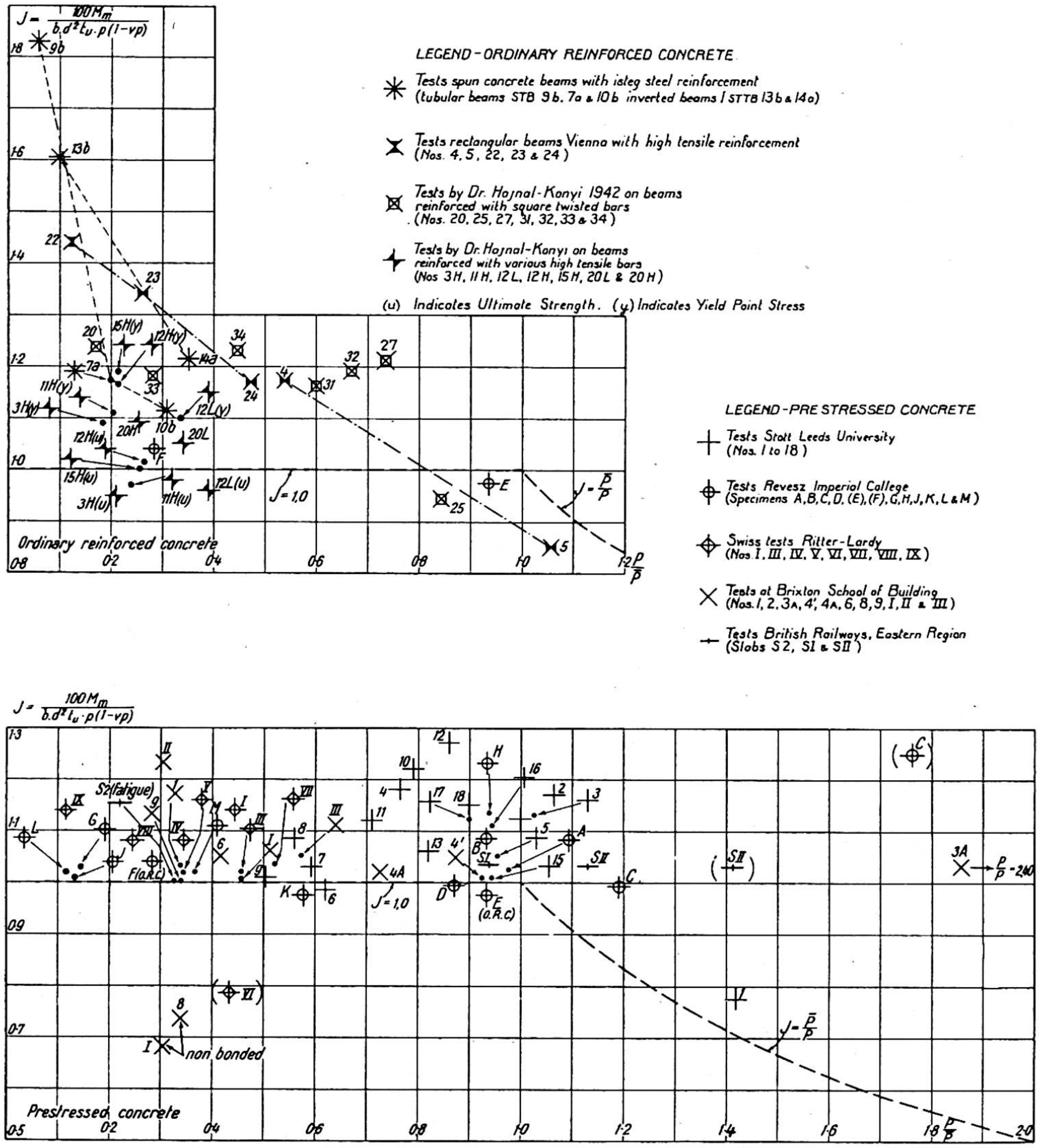


Fig. 7

the concrete strength was higher than stated in Table II. However, if a value $c_m = 3{,}000$ lb./in.^{2} is taken into account, a quite reasonable value for R is obtained, as seen from the diagram. It may be mentioned that this beam C contained bonded and additional non-bonded tensioned wires. Apparently this co-operation is quite successful, while the ultimate resistance is much less with beams with only non-bonded wires, as seen from specimens 8 and I of the Brixton tests. The behaviour of beams with non-bonded wires requires further study. Further tests have been carried out at the Imperial College and it is hoped results will be available to enable the author to supplement this study. It may be added that Prof. R. H. Evans was probably the first to point out the difference between the behaviour of bonded and non-bonded wires.[31]

Like the two beams with non-bonded wires the beam VI of the Swiss tests had a relatively low ultimate resistance. In this case the initial tensioning stress was about 50,000 lb./in.^{2} and only a small prestress may have remained effective after all losses had taken place. However, this cannot be taken as an explanation for a reduced ultimate resistance, since the present study has proved that beams with untensioned high-strength wires (E and F of tests by Revesz and 20L and 20H of tests by Dr. Hajnal-Konyi) reached approximately the same high values as prestressed beams. Moreover there are two beams with lower prestress among the tests by Revesz, i.e. G and H, which show no appreciable difference from the other results with high prestress, although one approached the balanced design. It seems therefore reasonable to exclude the test result VI of the Swiss tests. This may be also justified by the fact that for this test the lowest cracking load was obtained, but it would be very important for similar tests to be carried out to check this question.

It was previously mentioned that slab S2 was statically tested to failure after a fatigue test extended over three million repetitions. It may be pointed out that bonded 0·2-in. wire was provided and this test has proved the complete efficiency of this wire, although it was considered from tests [32] that wire of such a large diameter is not suitable. Apparently it depends greatly on the surface conditions, and the British wire of 0·2 in. diameter is suitable.

In conclusion it can be said that the tests presented have proved that generally the same conditions apply with regard to ultimate resistance to prestressed and non prestressed members. The modified simple formulae for ultimate resistance have shown a very good agreement with test values and when assessing the ultimate resistance safe results are obtained if a low stress c_m is taken, e.g. 0·6 c_u. The two charts, figs. 6 and 7, permit the separate investigation of concrete and steel resistance. This investigation is limited to bonded reinforcement and further research is necessary on the behaviour of non-bonded tensioned steel. In this case a reduction factor of, say, 0·60 to 0·80 will have to be considered and particulars will be shown in a supplement.

Acknowledgements

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Summary

The behaviour of concrete beams with high-strength reinforcement (including work-hardened steel) is investigated on the basis of various test results. Cracking is discussed and the resilience of prestressed concrete pointed out. Fatigue tests have shown that cracks which opened one million times closed entirely on removal of the load. The main study of the paper is devoted to ultimate load conditions. Formulae for the design are employed which allow a simple assessment of the ultimate resistance. These formulae are based on a rectangular stress distribution of the maximum concrete compressive stress c_m over a maximum depth of 0·5 d, resulting in a maximum resistance $M_{max}=0{\cdot}375\ bd^2c_m$. If this stress is taken as 0·6 c_u, a safe approximation of the ultimate resistance is obtained. The percentage for the balanced design $\bar{p}$ amounts to $0{\cdot}25/v$, where $v=t_u/200\ c_m$. The ultimate steel strength t_u is normally reached but must in certain cases be replaced by the yield-point stress t_y. Three charts are shown for the various test results. One contains the stress $Q=M_m/(bd^2)$, the other the stress-ratio $R=Q/c_m$, and the third the stress-ratio $J=t_m/t_u$. This gives a measure of the co-operation of the concrete tensile zone between the cracks and indicates the quality of adhesion between steel and concrete. The investigation has shown that the high-strength properties of steel and concrete can be fully exploited, both in prestressed and ordinary reinforced concrete, provided that efficient bond is ensured. If the reinforcement is not efficiently bonded, M_{max}, Q, R, and J are appreciably reduced and reduction factors must be considered.

Résumé

L'auteur étudie, sur la base de divers résultats expérimentaux, le comportement des poutres en béton armées avec de l'acier à hautes résistances, y compris l'acier écroui. Il aborde sommairement la question de la formation des fissures et attire particulièrement l'attention sur l'élasticité du béton précontraint. Des essais de fatigue ont montré que des fissures qui s'ouvraient sous une charge appliquée 1 000 000 fois se refermaient complètement au moment de la suppression de la charge.

La partie principale du présent rapport traite de la question de la rupture elle-même. L'auteur emploie des formules simplifiées pour la détermination approchée du moment de rupture. Ces formules sont basées sur une répartition rectangulaire de la contrainte maximum calculée dans le béton σ_{bmax} sur une hauteur maximum de 0,5 h, ce qui donne un moment maximum $M_{max}=0{,}375\ bh^2\sigma_{bmax}$. En admettant pour σ_{bmax} 60% de la résistance de cube W, on obtient pour le moment de rupture des valeurs approchées du côté correspondant à la sécurité. L'armature limite pour laquelle la rupture se produit par destruction soit du béton, soit de l'acier, est définie par $\mu_g=0{,}25/v$ avec $v=\sigma_{eB}/200\sigma_{bmax}$ en désignant par σ_{eB} la charge de rupture de l'acier, qui doit être ici remplacée dans certains cas par sa limite écoulement σ_{eF}.

Trois diagrammes mettent en évidence les résultats fournis par les essais. Ces diagrammes donnent:

la contrainte $Q=M_{max}/(bh^2)$
le taux de contrainte $R=M_{max}/(bh^2\sigma_{bmax})$
le taux des contraintes dans l'acier $J=\sigma_{emax}/\sigma_{eB}$, c'est-à-dire le rapport entre la contrainte calculée dans l'acier, pour une section de rupture et la charge de rupture de l'acier.

On obtient ainsi une mesure du degré de coopération entre le béton et l'acier, entre les fissures, ainsi qu'une mesure de l'adhérence entre ces deux éléments. Les recherches ici exposées ont montré que, sous réserve d'une bonne adhérence, les caractéristiques de résistance mécanique de l'acier et du béton sont pleinement utilisées, aussi bien dans le béton armé ordinaire que dans le béton précontraint. Si l'acier n'a pas une bonne adhérence M_{max}, Q, R et J sont reduits considérablement et l'utilisation des facteurs de reduction doit être considerée.

Zusammenfassung

Das Verhalten von mit hochwertigem (einschliesslich verdrilltem) Stahl bewehrten Betonbalken wird an Hand verschiedener Versuchsergebnisse untersucht. Die Rissbildung wird kurz behandelt und es wird auf die Elastizität von vorgespanntem Beton besonders hingewiesen. Ermüdungsversuche haben bewiesen, dass sich Risse, die sich unter wiederholter Belastung 1 000 000 mal öffneten, geschlossen haben, sowie die Last entfernt wurde. Der Hauptteil des vorliegenden Berichtes bezieht sich auf den Bruchzustand. Einfache Formeln werden verwendet, die eine angenäherte Bestimmung des Bruchmomentes ermöglichen. Diese Formeln sind auf eine rechteckige Spannungsverteilung der grössten rechnungsmässigen Betonspannung $\sigma_{b(max)}$ aufgebaut, die sich maximal auf die halbe Höhe h erstreckt, was ein Grösstmoment $M_{max}=0{\cdot}375\ bh^2\sigma_{bmax}$ ergibt. Wenn σ_{bmax} als 60% der Würfelfestigkeit W angenommen wird, dann ergeben sich Annäherungswerte für das Bruchmoment, die auf der sicheren Seite sind. Die Grenzbewehrung, bei welcher der Bruch entweder durch Beton- oder Stahlzerstörung erfolgt, ergibt sich als $0{\cdot}25/\nu$, $\nu=\sigma_{eB}/200\ \sigma_{b(max)}$, wobei σ_{eB} die Festigkeit des Stahles in einzelnen Fällen durch die Streckgrenze σ_{eF} zu ersetzen ist. Drei Diagramme zeigen die Ergebnisse der Auswertung der Versuche. Eines enthält die Spannung $Q=M_{max}/(bh^2)$, das andere das Spannungsverhältnis $R=M_{max}/(bh^2\sigma_{bmax})$ und das dritte das Verhältnis der Stahlspannungen $J=\sigma_{emax}/\sigma_{eBruch}$, d.i. das der rechnungsmässigen Stahlspannung für einen gerissenen Querschnitt und der Stahlfestigkeit. Dies stellt den Grad der Zusammenarbeit zwischen Beton und Stahl zwischen den Rissen dar und ist ein Mass für die Haftung zwischen Beton und Stahl. Die vorliegende Untersuchung hat bewiesen dass—gute Haftung vorausgesetzt-die hohen Festigkeitseigenschaften von Stahl und Beton sowohl beim gewöhnlichen Eisenbeton als auch beim vorgespannten Beton voll ausgenützt werden. In dem Falle, dass keine zuverlässige Haftung der Bewehrung gesichert ist, werden M_{max}, Q, R, und J wesentlich kleiner und Reduktionsfaktoren müssen eingesetzt werden.

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CII 2

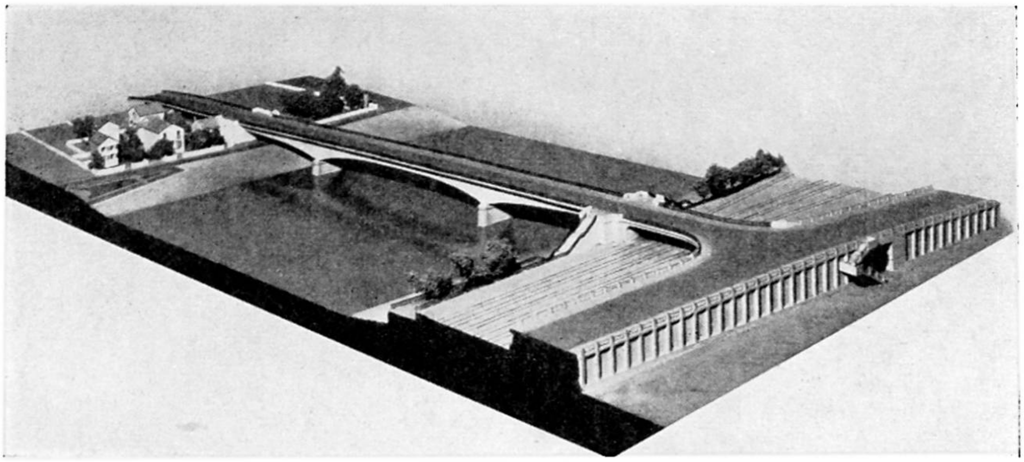
Le Pont de Villeneuve-Saint-Georges

The Villeneuve-Saint-Georges Bridge

Die Brücke von Villeneuve-Saint-Georges

HENRY LOSSIER et MICHEL BONNET

Ingénieur Conseil, Paris — Ingénieur des Ponts et Chaussées, Paris



Pont de Villeneuve-Saint-Georges sur la Seine Maquette

QUELQUES MOTS D'HISTORIQUE

Avant la guerre de 1939–1945, un vieux pont suspendu franchissait la Seine à Villeneuve-Saint-Georges, reliant les deux agglomérations de Villeneuve-Saint-Georges et de Villeneuve-le-Roi.

Cet ouvrage vétuste était insuffisant; une décision du Conseil Général du Département de Seine-et-Oise provoqua la mise au concours d'un nouvel ouvrage de conception moderne.

Le projet dressé par M. Henry Lossier, Ingénieur Conseil à Paris, fut retenu, et l'ouvrage—un pont cantilever en béton armé—fut construit de 1936 à 1939 par les soins des Etablissements Fourre' & Rhodes à Paris, sous la direction du Service Vicinal.

Mais le pont fut détruit presque aussitôt, en juin 1940, au cours des opérations militaires.

Après la guerre, il fallut envisager la reconstruction du pont. On décida de profiter de cette reconstruction pour tenter de mettre au point un nouveau type d'ouvrage en béton précontraint. Les études préliminaires furent confiées à M. Henry Lossier, Ingénieur Conseil. Le projet définitif fût dressé avec la collaboration étroite du Service Central d'Etudes Techniques du Ministère des Travaux Publics, et avec le concours de M. Demaret, Architecte en chef du Gouvernement pour la partie architecturale. M. Kellner, Ingénieur, assista M. Lossier.

Dans ce qui suit, nous désirons exposer quel est le principe du fonctionnement du nouvel ouvrage, quels en sont les principaux éléments constitutifs, et comment les Ingénieurs, en liaison avec l'Entreprise chargée de la réalisation des travaux (Ets. Fourre' & Rhodes à Paris) purent résoudre les problèmes essentiels de mise en œuvre posés sur le chantier à l'occasion de cette réalisation.

Caractéristiques générales retenues pour l'ouvrage

Description sommaire

L'ouvrage (fig. 1) est un cantilever à 3 travées mesurant respectivement 41 m., 78,20 m. et 41 m. de portée, la travée centrale comportant une partie indépendante de 39 m. 11 de longueur.

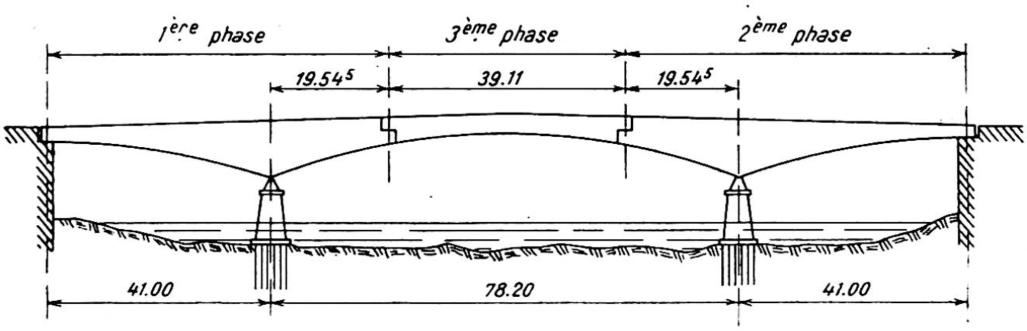


Fig. 1

Le tablier est constitué par une poutre évidée de 9 m. 25 de largeur totale, dont chacun des 3 caissons mesure 2 m. 85 de largeur libre.

La chaussée mesure 8 m. 40 et chacun des 2 trottoirs en encorbellement 2 m. 80 de largeur.

Le tablier repose:

- sur les 2 piles en rivière rehaussées de l'ancien pont, par l'intermédiaire d'articulations en béton fretté,
- sur les 2 culées, par l'intermédiaire de pendules également frettés.

L'épaisseur du tablier varie de:

- 2 m. 30 au milieu de la travée centrale,
- 6 m. 85 au droit de chaque pile,
- 2 m. 45 sur chaque culée.

La partie indépendante de 39 m. 11 repose sur une articulation à l'une de ses extrémités et sur un pendule à l'autre extrémité, ces dispositifs d'appui étant en béton fretté.

Dispositions particulières

Le tablier est en béton précontraint dans le sens longitudinal.

Cette précontrainte est réalisée par des câbles de ponts suspendus disposés à l'intérieur des caissons du tablier. Ces câbles sont accessibles et réglables en tous temps, des trous d'homme étant réservés dans la chaussée, ainsi que des ouvertures dans les parois verticales pour la circulation à l'intérieur de l'ouvrage.

Le passage des câbles à travers les voiles transversaux se fait par des trous prévus à cet effet.

Afin de supprimer tout glissement et tout frottement, l'infléchissement des câbles, dans la longueur courante de l'ouvrage, est réalisé par l'intermédiaire de balanciers en béton armé, placés devant les voiles transversaux, et articulés à leur pied.

Principe du fonctionnement de l'ouvrage

Conditions de fonctionnement

Le tracé et la tension des câbles sont déterminés de manière à satisfaire aux deux conditions principales suivantes pour chaque section:

1. Sous l'action de la charge permanente, la résultante des forces sollicitantes et des réactions des appuis doit passer, aussi exactement que possible, par le centre de gravité de la section, afin de ne provoquer qu'un moment fléchissant de très faible intensité. En d'autres termes, les contraintes longitudinales de compression du béton doivent être sensiblement uniformes sur toute la hauteur des diverses sections du tablier.

2. Sous l'action des cas de surcharges les plus défavorables, les contraintes longitudinales de compression ne doivent jamais être inférieures à 5 kg./cm.^{2} afin d'éliminer toute possibilité de fissuration.

La figure 2 représente le tracé des câbles.

Calculs de résistance d'une section

Considérons une section quelconque et désignons par (fig. 3):

G son centre de gravité
S sa surface
I son moment d'inertie
e la distance du centre G à l'extrados
i la distance du centre G à l'intrados
a l'angle de l'intrados avec l'horizontale
C_1 la traction des câbles horizontaux
C_2 celle des câbles obliques
c_1 la distance de C_1 au centre G
c_2 la distance de C_2 au centre G
b l'angle des câbles C_2 avec l'horizontale
N la composante normale de compression par les câbles
M_c le moment fléchissant engendré par les câbles
M_1 le moment fléchissant permanent dû aux forces extérieures
M_0 le moment fléchissant permanent total, soit M_c-M_1
M_2 le moment fléchissant accidentel maximum
R_E la compression de la fibre d'extrados
R_I celle de la fibre d'intrados

On a, en affectant le moment M_1 du signe moins:

$$M_c=C_1 \cdot c_1+C_2 \cdot c_2$$
$$M_0=M_c-M_1, \text{ supposé positif}$$

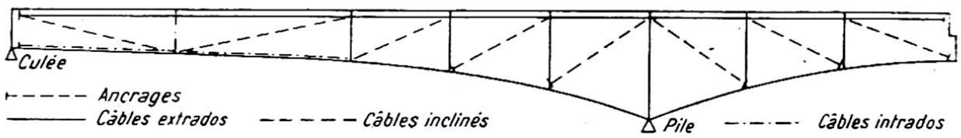


Fig. 2

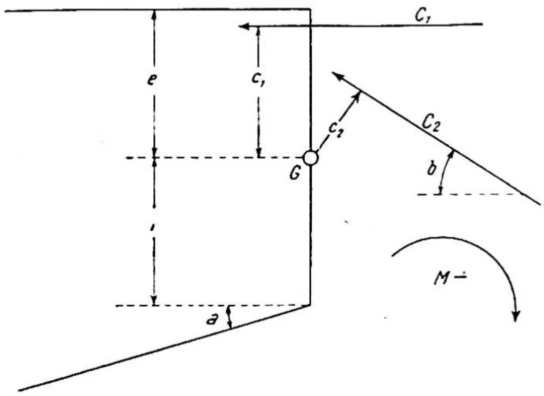


Fig. 3

Pour satisfaire totalement à la première condition, on devrait avoir $M_0=0$, et la section subirait alors une compression uniforme égale à N/S,
N étant égal à $N=C_1+C_2$ cosinus b.

Affectons les contraintes de traction du signe plus.

Compression permanente d'extrados: $R_E=-\frac{N}{S}-\frac{M_0 \cdot e}{I}$.

Compression permanente d'intrados: $R_I=-\frac{N}{S \cdot \cos a}+\frac{M_0 \cdot i}{I \cdot \cos a}$.

En raison de la grande élasticité relative des câbles par rapport au tablier en béton armé, ceux-ci n'interviennent que très faiblement sous l'action des charges accidentelles, leur contrainte n'étant alors majorée que de 1 à 2 kg./mm.²

Aussi, par excès de prudence, négligeons-nous leur action. On a donc, en supposant M_2 négatif:

Contrainte accidentelle d'extrados: $R'_E=+\frac{M_2 \cdot e}{I}$.

Contrainte accidentelle d'intrados: $R'_I=-\frac{M_2 \cdot i}{I \cdot \cos a}$.

Pour satisfaire à la première condition, on doit avoir:

$R_E-R'_E=$compression supérieure à 5 kg./cm.²

Par ailleurs, la contrainte d'intrados $R_I+R'_I$ ne doit pas excéder le taux-limite à la compression, fixé à 100 kg./cm.² dans le cas envisagé.

Les calculs de résistance aux *efforts tranchants*, qui font intervenir l'inclinaison des câbles pliés, celle de l'intrados et la compression longitudinale (cercle de Mohr), ne présentent aucune particularité.

Au droit de chaque nœud d'intrados, la résultante D de la traction C_2 des câbles inclinés et de la poussée B des balanciers est équilibrée par deux réactions:

l'une verticale V, intéressant le voile correspondant,
l'autre oblique W, intéressant le hourdis d'intrados,

ces deux éléments travaillant dans leur plan (fig. 4).

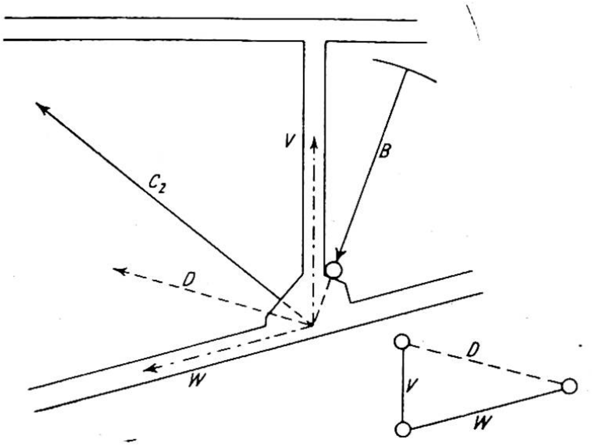


Fig. 4

Caractéristiques des principaux éléments singuliers

Câbles

Les câbles, spiraloïdaux, sont composés chacun de 193 fils de 41/10, de 160 à 180 kg./mm.^{2} de résistance à la traction. Ils supportent, en service permanent, un effort de 160 tonnes sous une contrainte de 63 kg./mm.^{2}, que l'action des surcharges peut porter à 65 kg. au maximum.

Ancrage des câbles dans le béton

Les câbles de précontrainte comportent, à chacune de leurs extrémités, un culot d'ancrage, conformément à la technique classique des ponts suspendus (fig. 5).

Le remplissage du culot est réalisé à l'aide d'un alliage ternaire: Plomb (83,7 %),

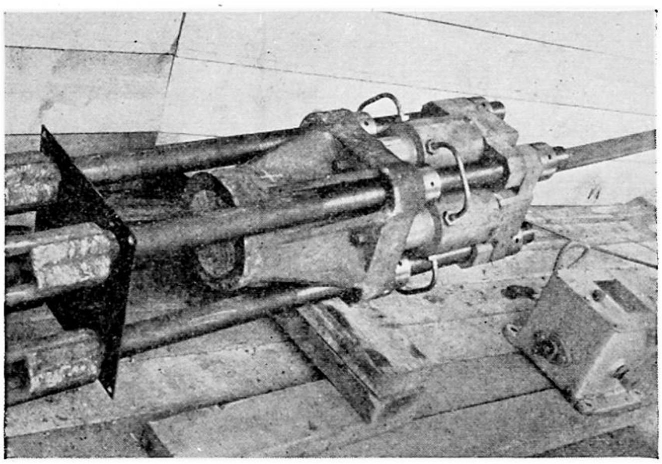


Fig. 5. Culot d'ancrage

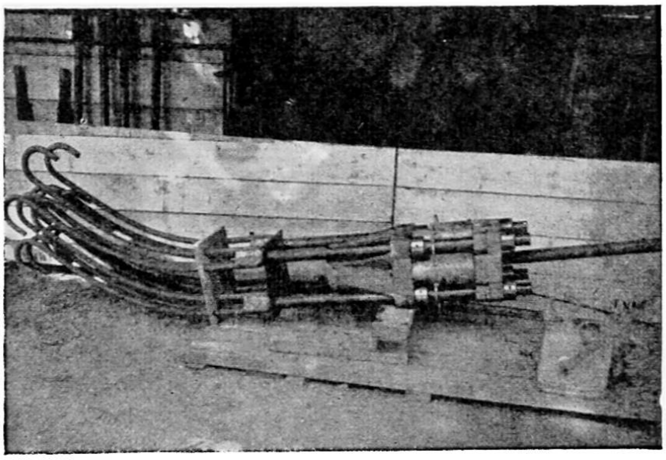


Fig. 6. Dispositif d'ancrage—Assemblages soudés $\phi 32$–$\phi 64$

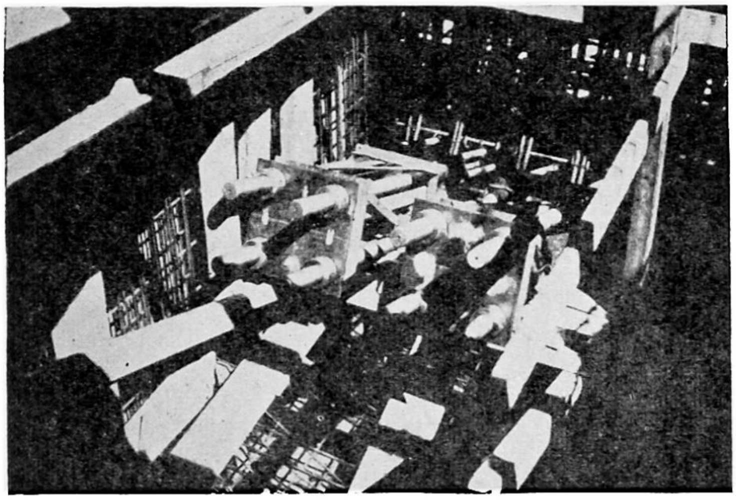


Fig. 7. Ancrages.—Mise en place dans le ferraillage d'un voile. On distingue les gabarits métalliques percés d'un trou central et permettant la mise en place correcte de l'ancrage par rapport à un fil métallique matérialisant l'axe du câble

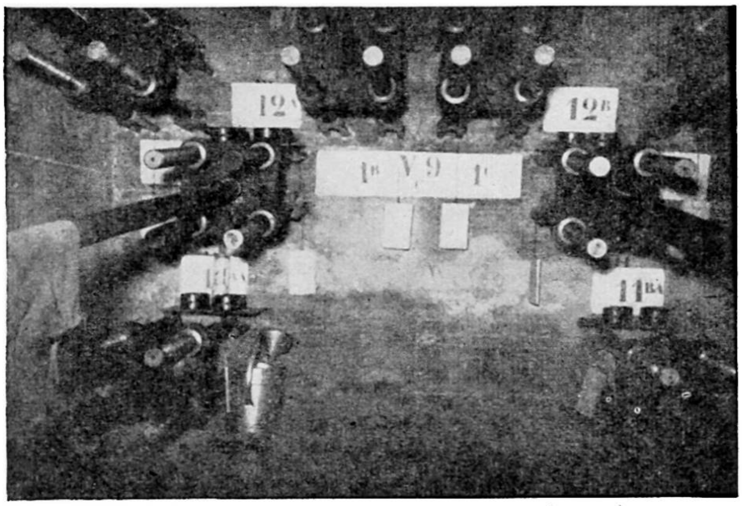


Fig. 8. Ancrages.—Voile 9 sur culée. Les numéros des câbles sont portés sur le voile, afin d'éviter les erreurs en cours d'exécution du programme de mise en tension

Antimoine (9,3 %), Etain (7 %), mis en œuvre en usine à 350° et conduisant à un coefficient de remplissage moyen de l'ordre de 98 %.

La fixation de culot (figs. 5 et 6) se fait par l'intermédiaire d'écrous sur 4 tiges de retenue filetées ϕ 64 mm., en acier mi-dur; chacune de ces tiges est elle-même soudée sur trois tiges d'ancrage ϕ 32 en acier mi-dur encastrées dans le béton de l'ouvrage (fig. 6).

L'épanouissement de chaque tige de retenue en trois tiges d'ancrage permet d'assurer dans de bonnes conditions la transmission des efforts de traction dans la masse du béton des voiles transversaux. Le cas échéant, on joue sur l'orientation des crosses, ainsi que sur la longueur et la courbure des tiges d'ancrage.

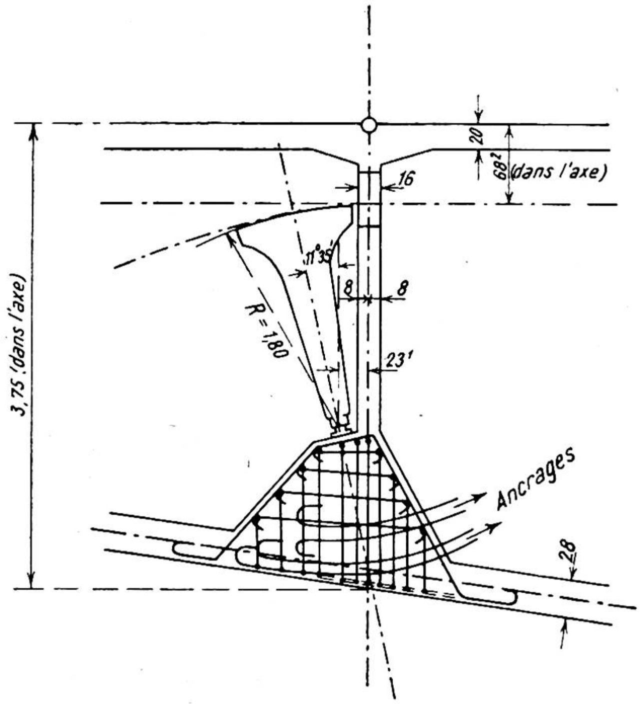


Fig. 9

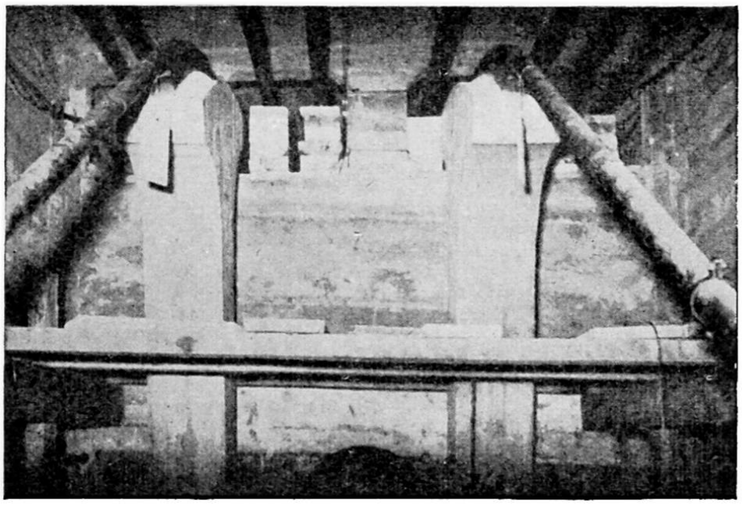


Fig. 10

Changement d'orientation des câbles-balanciers

Les changements d'orientation des câbles sont assurés à l'aide de balanciers.

Dans la travée centrale indépendante, la hauteur disponible pour les balanciers est faible, et l'on a utilisé des appareils en acier moulé.

Dans les consoles-culasses, où la hauteur disponible est beaucoup plus grande, on a adopté des balanciers en béton armé fortement fretté (voir figs. 9 et 10).

Les dimensions des profils des sellettes d'appui, des câbles et des surfaces de roulement des pieds de balancier, ont été définies compte-tenu des mouvements des divers balanciers lors des opérations de mise en tension des câbles. Une étude particulière a été faite pour chaque balancier.

PROCESSUS DE CONSTRUCTION

Phases d'exécution de l'ouvrage

L'importance de la navigation sur la Seine, à Villeneuve-Saint-Georges, exigeait que l'on maintint en permanence une passe marinière assurant le passage des convois montants et avalants.

Les Ingénieurs auraient désiré réaliser d'un seul coup l'ensemble du cintre. En fait, cette solution s'est avérée impraticable; en effet, le pont à construire étant en béton précontraint, les poutres n'auraient de résistance qu'après mise en tension des câbles: le cintre devant donc être capable de supporter toute la charge de l'ouvrage jusqu'à la mise en tension. Dans ces conditions, il fallait réaliser un cintre fort lourd et cette sujétion s'accommodait mal avec la nécessité de maintenir une passe marinière.

En définitive, les Ingénieurs se sont arrêtés à un processus de construction en plusieurs phases successives telle que, dans chaque situation, soit dégagée:

soit une passe unique supérieure à 35 m.,

soit deux passes (16 et 20 m.).

Ces phases sont schématisées sur la figure 11.

On a été conduit ainsi à réaliser d'abord les deux poutres consoles-culasses Rive Droite et Rive Gauche et, ensuite seulement, grâce à un cintre suspendu, la travée indépendante centrale.

Conséquences, en ce qui concerne la mise au point du programme de mise en tension des câbles

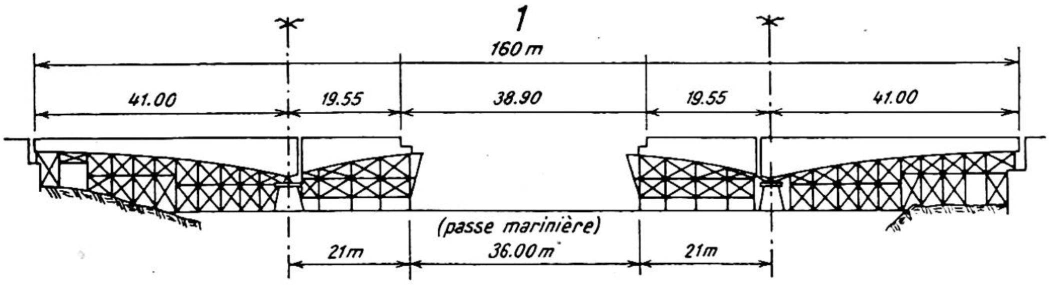
Les Ingénieurs ont dû préciser dans quelles conditions la tension des câbles de précontrainte de l'ouvrage serait portée de 0 à 160 tonnes (tension de service).

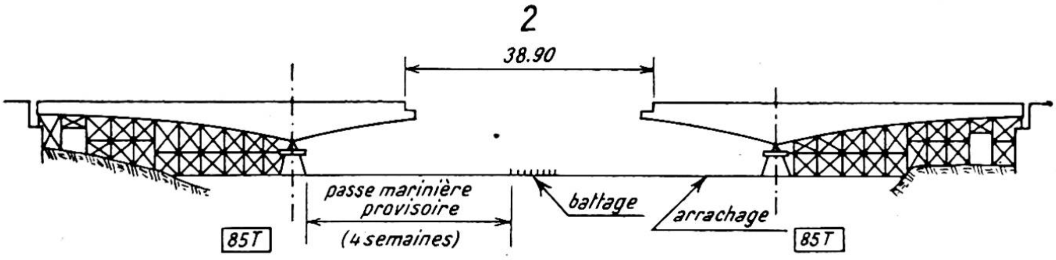
Il a été nécessaire de tenir compte, dans cette étude, des étapes successives de construction de la travée indépendante centrale: au fur et à mesure de l'avancement des travaux, la charge P en about de console augmente, pour atteindre finalement la charge figurant dans la note de calcule (P=489 tonnes). La tension des câbles des poutres consoles-culasses doit être réglée en conséquence, de manière à éviter sous les divers états intermédiaires de charges en about de console, la création de contraintes anormales dans le béton des voiles longitudinaux, d'intrados et d'extrados.

(*a*) Paliers de mise en tension

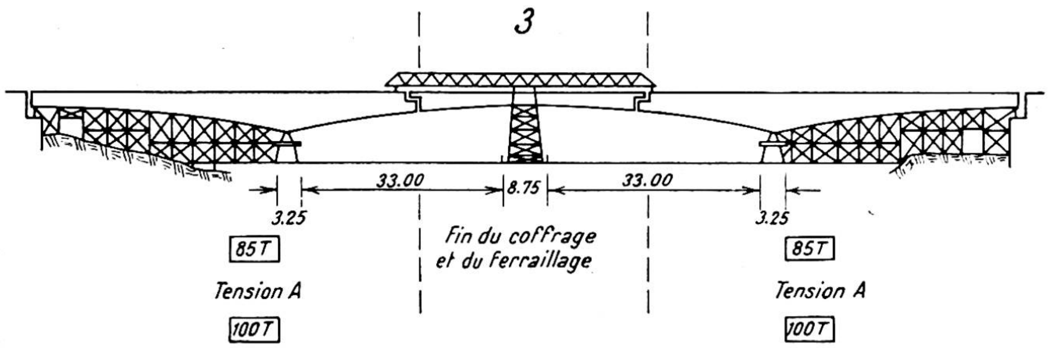
L'étude a été faite sur un diagramme comportant:

- en abcisses, les charges successives C de mise en tension des câbles (comprises entre 0 et 160 tonnes);
- en ordonnée, les charges P en about de console, aux divers stades de la construction de la travée indépendante centrale.





Battage des pieux et échafaudages des consoles-culasses-Coffrage-Ferraillages. Coulage du béton des 2 consoles-culasses en ménageant un joint de 0 m. 75 à 0 m. 50 de l'axe des piles, exécuté en dernier lieu-mise en place des câbles et tension des câbles jusqu'à 85 tonnes.



Enlèvement des échafaudages et des pieux sous consoles. Battage des pieux de la passe marinière

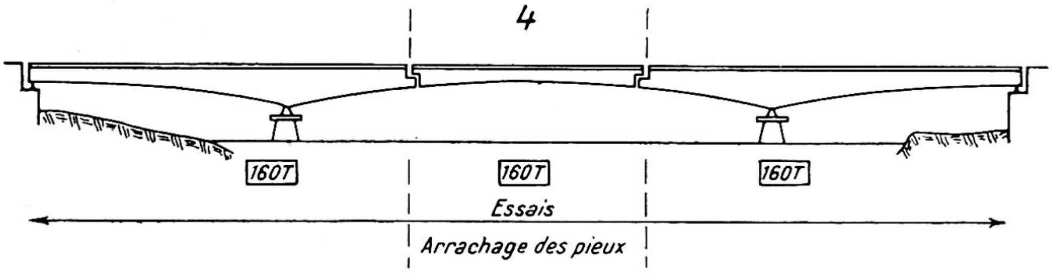


Fig. 11

La traduction analytique pour les diverses sections des poutres consoles des conditions suivantes:

voile d'intrados comprimé $R \geqslant 5$ kg./cm.²
voile d'extrados comprimé $R \geqslant 5$ kg./cm.²
taux de cisaillement dans le béton inférieur à 20 kg./cm.² en tout point,

a conduit à limiter, dans le plan (P, C) un domaine de variation tel que les états d'équilibre correspondants ne provoquent l'apparition d'aucune contrainte anormale dans les voiles des poutres consoles-culasses (le cintre sous culasse étant supposé maintenu jusqu'aux derniers stades de la construction).

Les Ingénieurs ont défini ainsi, pour les câbles des poutres consoles-culasses, un certain nombre de "paliers" successifs de mise en tension, correspondant aux diverses phases de construction de la travée indépendante centrale.

Ces paliers se traduisent sur la graphique (P, C) par un diagramme en escalier, compris dans le domaine de variation précédemment défini (voir fig. 12).

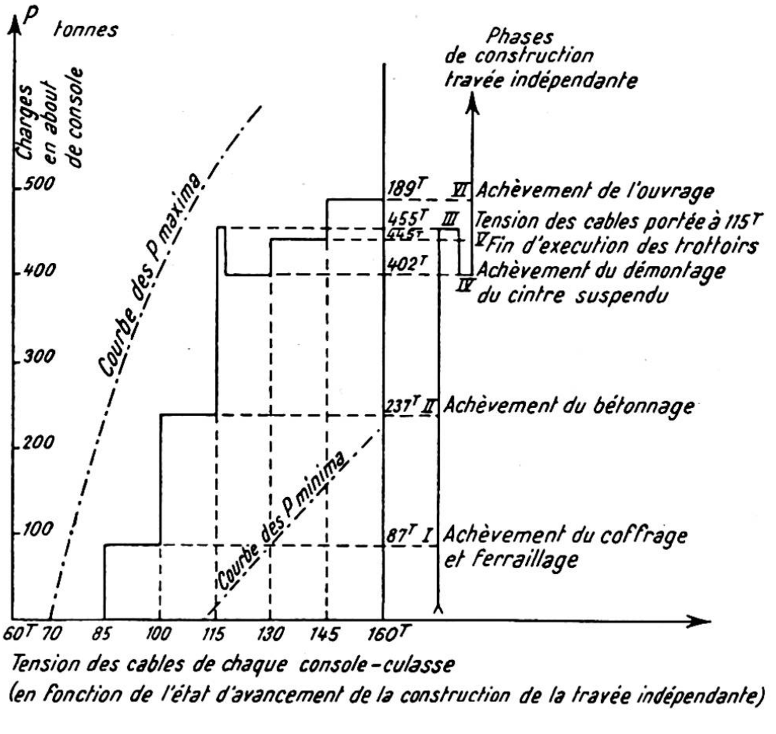


Fig. 12. Diagramme de mise en tension des câbles des poutres consoles culasses

Une étude analogue a été faite pour la mise en tension des câbles de la travée indépendante centrale.

(b) Etapes de mise en tension

Pour passer d'un palier de mise en tension au suivant dans le cadre du programme défini ci-dessus, les Ingénieurs ont adopté des "étapes" uniformes de 15 tonnes.

Soit T_0 la tension de l'ensemble des câbles d'un élément d'ouvrage dans un état intermédiaire du programme. Le processus conduisait à porter la tension de tous les câbles de l'élément considéré, de T_0 à (T_0+15) tonnes, puis de (T_0+15) à (T_0+30) tonnes, et ainsi de suite.

Au sein d'une même "étape" de 15 tonnes, l'ordre de mise en tension des câbles d'un même élément d'ouvrage est demeuré immuable; il avait été déterminé une fois pour toutes, compte tenu de la nécessité d'éviter l'apparition de contraintes anormales dans les voiles transversaux.

(*c*) Sens de mise en tension—Mouvements des balanciers

Compte tenu des dispositions générales qui précèdent, les Ingénieurs ont procédé à une étude détaillée des mouvements des divers balanciers au cours de la mise en traction des câbles. Cette étude a permis de définir la position initiale à donner à chacun des balanciers, et les sens de traction à adopter (tantôt côté "rive gauche" et tantôt côté "rive droite") pour que, en définitive, sous la tension de service (160 tonnes), les balanciers occupent leur position théorique (axe de chaque balancier confondu avec la bissectrice de l'angle des directions de câble).

Procédant ainsi, il a été possible, à ce jour, de mettre en tension de 10 à 120 tonnes, dans de bonnes conditions, et sans difficultés notables, les $(39 \times 2) = 78$ câbles des deux poutres consoles-culasses, en utilisant seulement 4 jeux de 4 vérins.

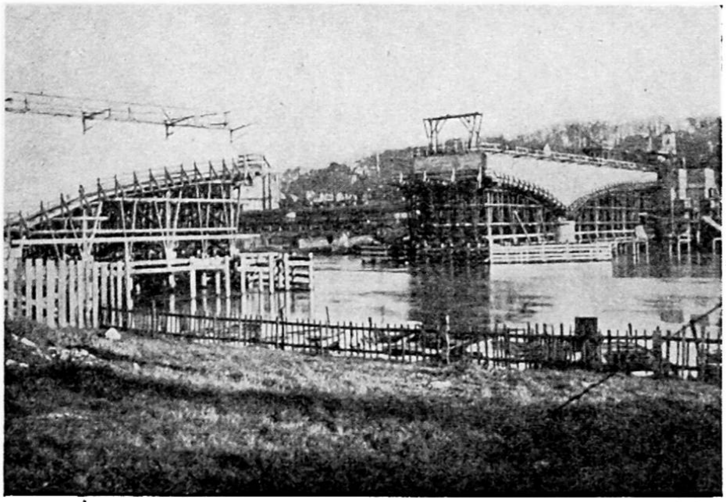


Fig. 13. Câbles tendus à 85 tonnes. On va procéder à l'enlèvement du cintre sous console-culasse Rive-gauche

(*d*) Dispositif de mise en tension

La mise en tension de chaque câble s'effectue à l'aide d'un jeu de 4 vérins, disposés entre la plaque d'amarrage du culot, et une plaque d'appui fixe solidaire des tiges de retenue (figs. 14 et 15).

Les manomètres de contrôle des pressions sont étalonnés chaque jour sur le chantier, à l'aide d'une balance à fléau spéciale.

Contrôles en cours d'exécution

Les taux de travail adoptés dans la note de calculs de l'ouvrage, sont les suivants:

Béton: à la compression . . . . 100 kg./cm.^{2}
au cisaillement . . . . 10 ,,
(porté à 25 kg./cm.^{2} si les armatures transversales sont calculées pour résister seules aux efforts tranchants).
à l'adhérence . . . . 10 kg./cm.^{2}
(porté à 20 kg./cm.^{2} si les barres sont munies de crosses).

Acier clair (câbles): . . . . 65 kg./mm.^{2}

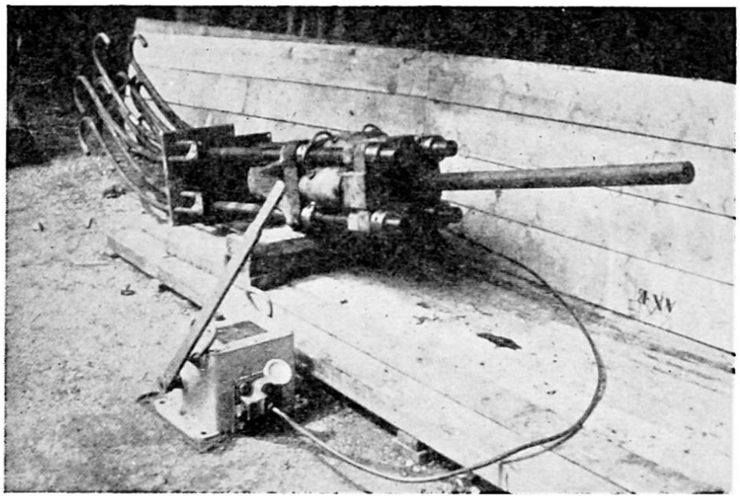


Fig. 14

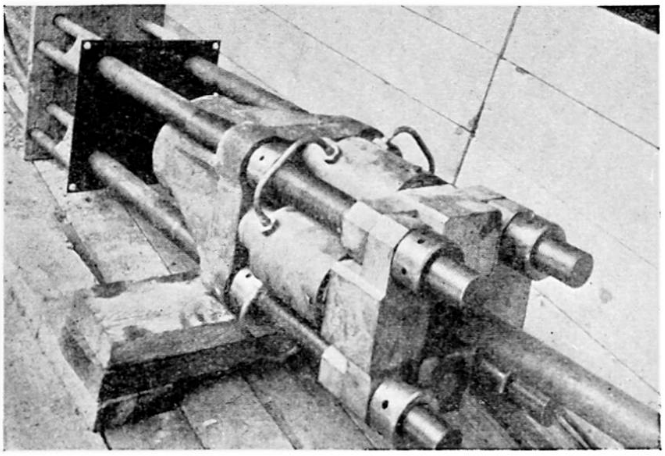


Fig. 15

Ces taux étant relativement élevés, un contrôle sévère de la qualité des matériaux mis en œuvre et de la résistance des assemblages a été assuré en cours d'exécution.

Le béton mis en œuvre dans les éléments de la poutre était constitué comme suit (pour un mètre cube de béton):

sable 0/8 des sablières de la Seine	285 litres
sable 0/2 de Pitre (Rouen)	57 ,,
gravillon 5/25 des sablières de la Seine	885 ,,
ciment 250/315 C.P.A.	400 kg.
eau totale	210 litres

Toutes précautions ont été prises en vue d'assurer l'homogénéité des fournitures de ciment et la constance du dosage en eau.

Des prélèvements systématiques de béton ont été effectués, à raison de 12 éprouvettes prismatiques $10 \times 10 \times 30$ et 9 éprouvettes cubiques $20 \times 20 \times 20$ par 10 mètres cubes de béton mis en œuvre. Chaque série d'éprouvettes était prélevée sur une même gachée, les modalités de fabrication et de conservation des éprouvettes avaient été définies avec précision; celles-ci ont fait l'objet d'essais à 7, 28 et 90 jours.

L'ensemble des résultats expérimentaux relatifs à ces essais est en cours de dépouillement; il permettra de vérifier certaines hypothèses relatives aux lois de répartition des observations faites et donnera, par ailleurs, des indications intéressantes sur le comportement de deux séries homologuées d'éprouvettes de formes différentes (prismatique et cubique).

Les résultats acquis à ce jour pour les poutres consoles-culasses sont les suivants (résistance à la compression à 90 jours):

Désignation	Unité	Eprouvettes $10 \times 10 \times 30$		Eprouvettes $20 \times 20 \times 20$	
		Console-culasse		Console-culasse	
		Rive gauche	Rive droite	Rive gauche	Rive droite
Nombre d'éprouvettes	—	244	252	156	207
Moyenne arithmétique	kg./cm.^{2}	347	359	440	444
Ecart moyen arithmétique	kg./cm.^{2}	66,1	37,6	43,2	32,3
Ecart moyen quadrat	kg./cm.^{2}	84,9	48,0	56,8	42,5

Toute l'organisation du chantier de bétonnage est axée vers la recherche de la dispersion minima dans les résultats d'essais.

Acier clair (câbles)

Les conditions imposées pour la réception des câbles en usine sont celles de l'Instruction du 15 octobre 1947 sur les téléfériques à voyageurs.

Les caractéristiques générales des câbles sont les suivantes:

diamètre nominal 66 mm.
composition 193 fils 41/10—type spiraloïdal monotoron
acier de 160/180 kg./mm.^{2} de résistance à la rupture
angle de câblage 20°

Chaque câble, muni de ses culots d'ancrage, a été mis en prétension à 1,25 fois la charge de service, sur le banc d'essai du chantier. Cette opération a permis de contrôler la bonne tenue de tous les culottages et de stabiliser le câble en provoquant la mise en place définitive des fils les uns par rapport aux autres.

Six essais à la rupture ont été effectués sur la machine de traction de 500 tonnes des Laboratoires de la Marine à Paris; les échantillons soumis aux essais étaient constitués par une longueur de 1 m. 40 de câble, culottés à leurs deux extrémités à l'aide de culots identiques à ceux adoptés sur l'ouvrage, et maintenus sur la machine grâce à 4 tiges d'ancrage ϕ 64 mm.

Les essais ont permis de vérifier la tenue de l'ensemble du dispositif d'ancrage—câble, culots, barres de retenue.

Dans tous les cas, c'est le câble qui s'est rompu; les taux observés ont été les suivants (charge de service 160 tonnes):

Echantillon N°.	1	2	3	4	5	6
Charge de rupture (tonnes)	420	407	398	400	394	415

Les essais ont également permis d'examiner le comportement élastique des câbles et ont conduit les Ingénieurs à adopter comme allongement élastique sous 160 tonnes après prétension à 200 tonnes: $de/e = 4{,}25\,‰$. Cette constante a été utilisée dans l'établissement du programme de mise en tension.

Ancrages: contrôle des soudures

Des prélèvements systématiques d'assemblage soudés (3 barres ϕ 32 soudées sur une barre ϕ 64) ont été effectués en cours d'exécution des travaux (un assemblage par lot de 50).

Les essais de rupture, effectués sur la machine de traction du Laboratoire du Conservatoire National des Arts et Métiers à Paris, ont donné des résultats satisfaisants (rupture en dehors des soudures, pour des taux de travail de l'ordre de 55 kg./mm.^{2}).

On a procédé à des essais sur modèle réduit en vue de fixer les dispositions à adopter pour le ferraillage des dalles d'ancrage disposées aux extrémités de la travée indépendante centrale (essai sur dalle réduite au $\frac{1}{3}$).

Toutes dispositions ont été prises en vue de permettre le contrôle des contraintes dans l'ouvrage en cours de mise en tension et dans l'ouvrage terminé:

(*a*) *Béton:* 100 extensomètres acoustiques à corde vibrante (Société des Télémesures Acoustiques à Paris) ont été disposés dans le béton des voiles longitudinaux et transversaux de l'ouvrage. Ils permettent de suivre les opérations de mise en tension, et de comparer à chaque instant les contraintes observées à celles prévues dans la note de calcul.

(*b*) *Câbles:* des bases de mesures extensométriques (colliers) ont été disposées sur tous les câbles de l'ouvrage, et permettent un contrôle rapide des tensions, indépendamment des indications fournies par les manomètres.

CONCLUSIONS

Le chantier de reconstruction du Pont de Villeneuve-Saint-Georges, situé à proximité de Paris, constitue un exemple de "Chantier Laboratoire" sur lequel les Ingénieurs peuvent procéder à divers essais présentant un caractère d'intérêt général.

Le principe de fonctionnement de l'ouvrage, dont les avantages ont été soulignés plus haut, impose certaines sujétions d'exécution incontestables. Il exige, en particulier, une extrême précision dans la mise en place des ancrages, et un contrôle très poussé des opérations de mise en tension.

Ces sujétions lors de la construction sont compensées par la suite; en raison de la facilité avec laquelle les Ingénieurs pourront suivre l'évolution de la précontrainte, et compenser, si cela s'avère nécessaire, les effets du retrait, du fluage, et de la relaxation de l'acier.

Une telle réalisation constitue, enfin, un bon exemple de la collaboration fructueuse entre: Administration, Technicien Privé, Architecte et Entreprise.

Résumé

Le pont de Villeneuve-Saint-Georges franchit la Seine, près Paris, par 3 travées mesurant respectivement 41 m., 78,20 m. et 41 m. de portée. Son tablier, du type cantilever, comporte une partie indépendante de 39,11 m. au milieu de la travée centrale.

Il est précontraint dans le sens longitudinal par un système souple de câbles de ponts suspendus et de balanciers. Ce système, règlable en tous temps de l'intérieur de l'ouvrage, permet de compenser entièrement, à la demande, les effets de détente des câbles engendrés par le retrait et le fluage du béton, ainsi que par la relaxation propre de l'acier.

Le tracé et la tension des câbles sont déterminés de telle sorte que, sous la charge permanente, les diverses sections du tablier ne subissent que des efforts de flexion relativement très faibles, aucune traction du béton ne se produisant sous l'action des surcharges les plus défavorables.

L'exécution de cet ouvrage a été précédée par une série d'essais élémentaires concernant la résistance des câbles, de leurs culots, des soudures, des plaques d'ancrage en béton armé, etc.

Les diverses phases de la mise en œuvre du tablier sont également exposées.

Summary

The Villeneuve-Saint-Georges bridge crosses the Seine near Paris in 3 spans of 41 m., 78·2 m. and 41 m. In the centre a girder of the cantilever type, 39·11 m. long, is suspended.

The bridge is prestressed longitudinally with flexible cables on rocking bearings. This system, which can be adjusted at any time within the bridge, allows for compensating for the loss of tension due to shrinkage and creep of the concrete and plastic deformation of the cables.

The positioning and the tensioning of the cables are adjusted in such a way that the various sections of the roadway are subjected only to slight bending stresses under the permanent load and no tensile stress occurs in the concrete, even under the most unfavourable distribution of the effective loads.

Before the bridge was built numerous tests were made regarding the carrying capacity of the cables, their anchorages, the weld positions, the reinforced-concrete anchor plates, etc.

The various stages during the progress of the work are described in the paper.

Zusammenfassung

Die Brücke von Villeneuve-Saint-Georges überspannt die Seine bei Paris in 3 Öffnungen von 41 m., 78,2 m. und 41 m. Spannweite. Im Mittelfeld ist ein Träger von 39,11 m. (Typus Cantilever) eingehängt.

Die Brücke ist in Längsrichtung mit biegsamen Kabeln auf Pendellagern vorgespannt. Dieses im Innern der Brücke jederzeit regulierbare System erlaubt, die infolge Schwinden und Kriechen des Betons sowie infolge der plastischen Verformung der Kabel entstandenen Spannungsverluste auszugleichen.

Die Führung sowie die Spannung der Kabel sind so bestimmt, dass unter der ständigen Last die verschiedenen Querschnitte der Fahrbahn nur kleine Biegebeanspruchung erleiden, und auch bei der ungünstigsten Anordnung der Nutzlasten keine Zugbeanspruchung im Beton entsteht.

Vor der Ausführung wurden zahlreiche Versuche über die Tragfähigkeit der Kabel, ihrer Verankerungen, der Schweiss-Stellen, der Ankerplatten in Eisenbeton, usw. durchgeführt.

Die verschiedenen Bauetappen sind im Bericht ebenfalls beschrieben.

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CII 2

Continuity in prestressed concrete

La continuité dans le béton précontraint

Die Kontinuität im vorgespannten Beton

PROF. G. MAGNEL
Ghent

INTRODUCTION*

All those who have been pioneers in the field of prestressing have started by making exclusive use of simply supported beams; and they were quite right, as it was necessary to become thoroughly acquainted with the new technique by first applying it to the easiest case.

However, from the very beginning, the necessity has been felt for its application to statically indeterminate structures, and, indeed, it is unavoidable in many cases, as for example:

(*a*) the construction of multi-storey buildings;
(*b*) the construction of bridges with two or more spans; particularly when the spans are large and the height available for the bridge deck at midspan is very reduced, while the available height is much greater above the intermediate supports.
(*c*) the construction of buildings, even with only one storey, in areas subject to earthquakes.

In addition the desire to make use of continuity arises from the fact that it is a way to economise in anchorages and, consequently, to make prestressed beams with short spans economically; even in the case where anchored cables are used.

THE DIFFICULTY OF THE PROBLEM

Many difficulties were met by those who tried to apply prestressing to statically indeterminate structures. The following difficulties are worth mentioning:

(*a*) The method of design is not at first sight straightforward, although it is seen

* The word "prestressing" is taken to mean "stressed previous to the live load acting on it" and no difference is made between what in England are called "prestressing" and "poststressing."

immediately that it does not involve any new principles. Several specialists have published their methods and all that can be said is that they are all equivalent—being nothing else than the application of Hooke's law—and that the best is the one which one knows best and which one has applied many times.

(*b*) What is worth mentioning is that a rather small accidental displacement of the cable in a continuous beam—and this is also true for all statically indeterminate structures—produces an important variation in the external moments due to prestressing which the author has called the secondary moments.

Take, for example, continuous beams with three or two spans of 49 ft. each, calculated as shown in the author's book:* the secondary moments at the internal supports are the following:

Eccentricities	Case A	Case B
	Three spans	
At end support	0	0
At middle of end span	−8·5 in.	−9·5 in.
At middle of midspan	−3·3 in.	−3·3 in.
At interior support	+8·0 in.	−8·0 in.
Secondary moment at interior support	−27,100 lb.-in.	+8,300 lb.-in.
	Two spans	
At end support	0	0
At midspan	−10·1 in.	−11·0 in.
At mid-support	+0·9 in.	0 in.
Secondary moment at interior support	+837,000 lb.-in.	+975,000 lb.-in.

It will be seen that in the case of three spans the secondary moment changes considerably (even its sign changes) for a difference of 1 in. in the eccentricity of the cable; however, this is not important, as the absolute value of the secondary moment is in this case almost negligible.

In the case of two spans a difference of less than 1 in. in the exact position of the cable produces a difference of 16% in the value of the secondary moment, to which corresponds a variation in stress of 143 lb./in.^{2} in the beam, or about 7% of the permissible stress. It may be concluded from this that the exact position of the cable is of great importance and that it is the duty of the designer to see which is the best position and shape of the cables.

(*c*) The execution of statically indeterminate structures is not free from special difficulties, of which only three are discussed below:

(i) There is the frictional resistance of the cables in their housing; the loss of prestressing force corresponding to this friction exists also in the case of simply supported beams with curved cables, but is less important, as the cable has generally only one curvature. In continuous beams it is sometimes easy for the designer to use cables with several changes of sign of the curvatures and in these cases the loss through friction cannot be considered to be negligible. The ideal solution is to use straight cables, or cables only deviated at one point, being straight between this point and the ends.

* G. Magnel, *Prestressed Concrete*, p. 102, fig. 75, Concrete Publications, London.

However, this is not always possible. The author has made some tests on losses through friction and the results of the tests are given in an appendix.

(ii) There is the tendency for designers to achieve continuity by using short cables placed in the beam above the internal supports, with their ends protruding from the underside of the beam not far from the columns. The author does not think this is an arrangement that can be recommended except in cases of large spans. It is indeed, according to the author's experience, impossible to prestress short cables with sufficient accuracy. Take a cable of 15 ft. long to be prestressed up to 140,000 lb./in.^{2}, this means a total elongation of about 0·90 in.; in other words, by working with one jack at each end of the cable—as has to be done to decrease the frictional loss—the elongation to be produced by each jack is about 0·45 in. If it is remembered that in all known systems, where the wires are fixed in pairs or in larger numbers by a wedge, the difference from one case to another in slipping of the wires when (the wedge being driven home) they are released, amounts to 0·10 in., it is seen that the prestressing cannot be done with an accuracy of more than 22%, which the author considers as insufficient; moreover the error due to ignorance of the exact friction loss has to be added to this. In the present state of development, any other means of establishing continuity by means of short elements above the columns cannot be foreseen except through the use of steel bolts with a high elastic limit, the fixing device being a nut or something equivalent.

(iii) There is the difficulty in connection with expansion and contraction joints, where a special arrangement has to be developed to allow for the prestressing operation. This special arrangement is of course dependent upon the special kind of structure to be made. If the case of framed buildings is considered, two solutions are possible: Either make three spans continuous with a cantilever extending to about one-fifth of the fourth span; then leave the fourth span open in its central part and make the spans 5, 6 and 7 continuous with a short cantilever extending through about one-fifth of the fourth span. The ends of the two cantilevers in the fourth span can then be bridged over by a prefabricated beam in prestressed concrete simply supported on the cantilever ends. Or, alternatively, build the central part of the fourth span in ordinary reinforced concrete, provided dowel bars have been placed in the ends of the cantilevers.

A similar arrangement is going to be applied in Belgium in the case of a very important mushroom slab. It is intended to build the mushroom panel above the columns, using the prestressing technique in two directions; the remaining parts of the slab will be in ordinary reinforced concrete.

General remarks

The author would like to emphasise that in his opinion one should not be too afraid of the above difficulties. It must not be forgotten that the computations of stresses to be made are very inaccurate, as they are based on the elastic theory. Concrete has the great property of adapting itself to different conditions, mainly in statically indeterminate structures. Moreover, the factors of safety permitted for concrete are generally very high, and the concrete in prestressing work is not only made better than in ordinary reinforced concrete, but is cured under better conditions, and some of its hardening is done under pressure.

The main point, in the author's opinion, is to be sure of the value of the prestressing forces given by the steel at the time of prestressing and this should and can be done with an accuracy of about 5%.

Consequently, calculations which take too much time should not be made with the view of achieving better accuracy; let the designer concentrate on a good general conception of the structure to be made and see to it that the prestressing operation is done in the most perfect way.

SOME EXAMPLES OF CONTINUITY IN BELGIUM

Figs. 1, 2, 3, 4 and 5 give the general arrangement of framed buildings in

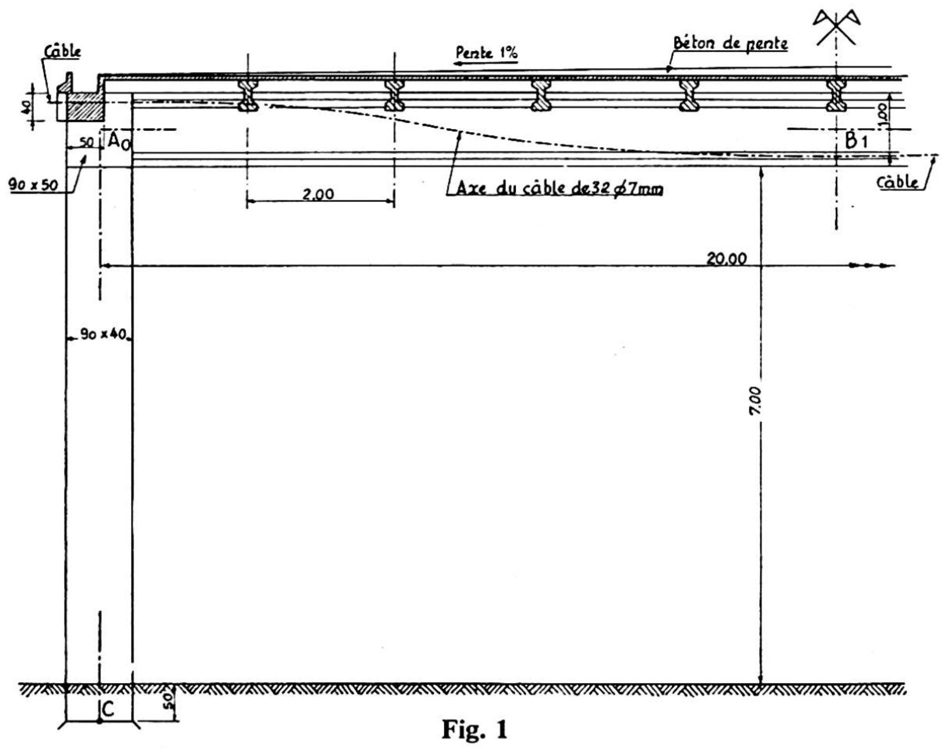


Fig. 1

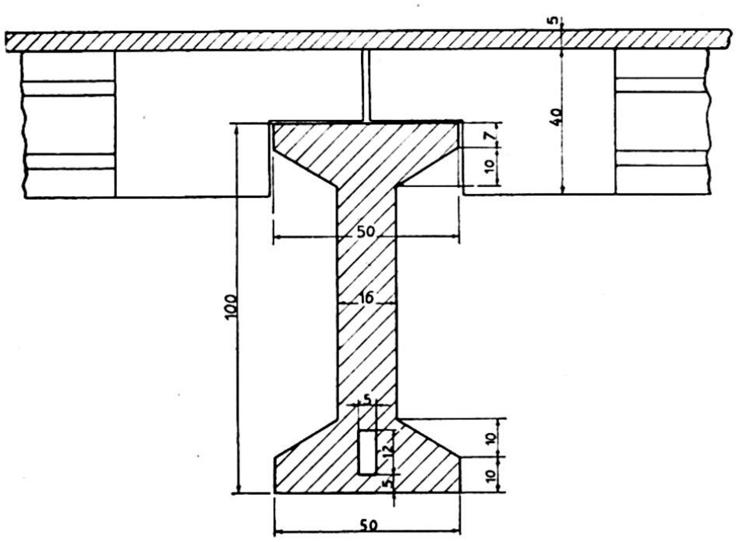


Fig. 2

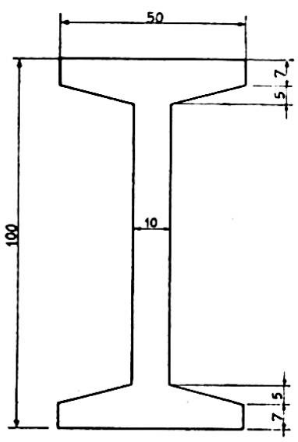


Fig. 3

prestressed concrete. Figs. 1, 2 and 3 show the case of a simple frame of 66 ft. span. Two possibilities are shown;

(i) the beam is either made monolithic with the cable hidden in it (fig. 2), or
(ii) the beam is made in prefabricated blocks with the cables placed at each side of the web (fig. 3).

In both cases the tops of the columns serve as end blocks for the beam. The columns are either in ordinary reinforced or in prestressed concrete. Fig. 4 shows a one-span multi-storey building. Fig. 5 shows a multi-span one-storey building.

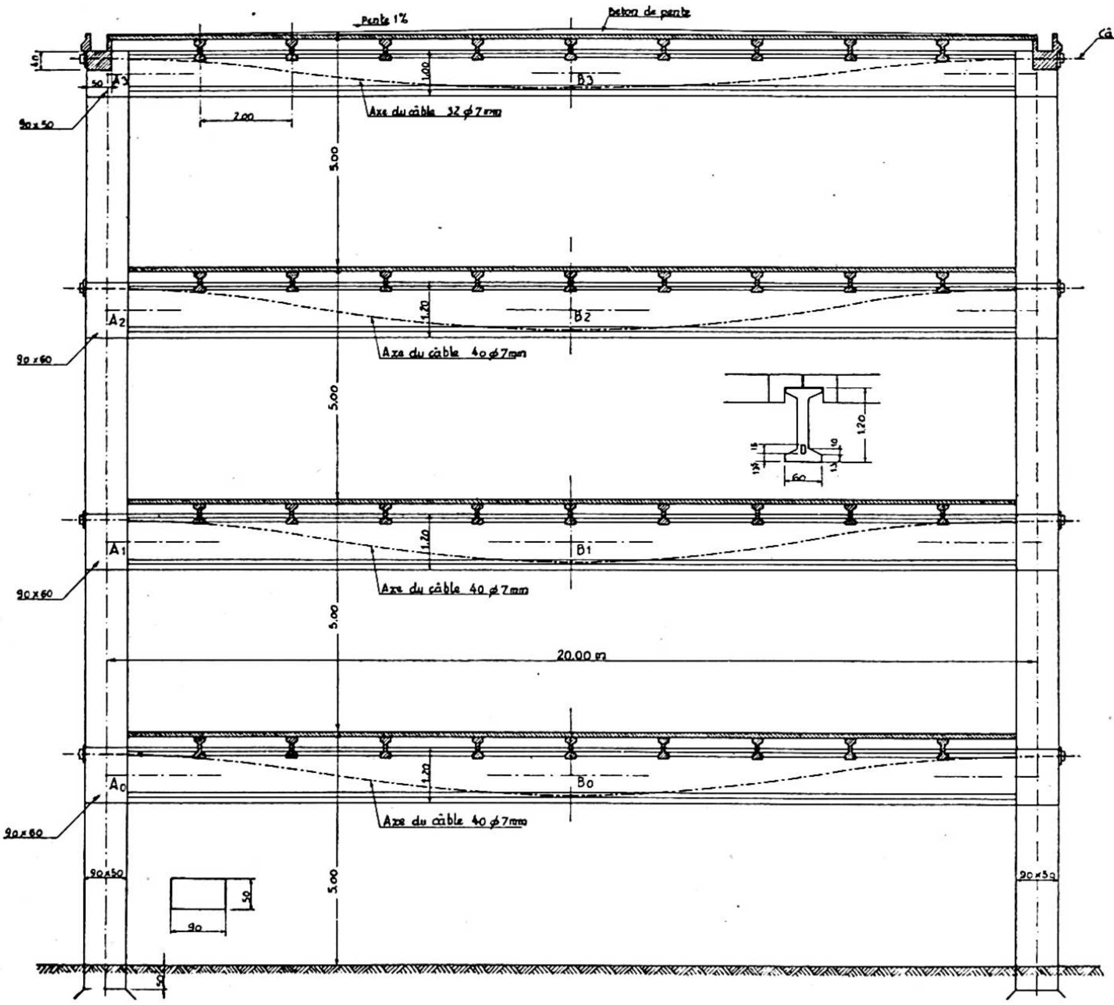


Fig. 4

Three examples of structures that have actually been built are given below:

(*a*) A two-storey building built in a contractor's yard; it was a much needed building, but it was decided to make it in prestressed concrete frames as a first experiment in this new direction (fig. 6). The span of the frame is about 53 ft.; the beams are made in prefabricated blocks, the cross-section of which is shown on fig. 6; the cables are placed outside the web on each side. Figs. 7 and 8 show some aspects of the structure.

(*b*) A four-storey office building built at Leopoldville in the Belgian Congo (fig. 9).

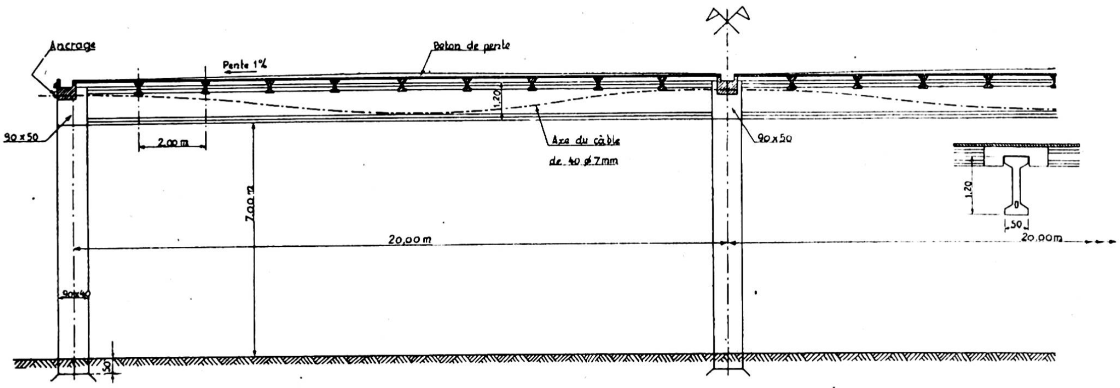

Ancrage
Pente 1%
Beton de pente
90 x 50
2,00 m
1,20
Axe du câble
de 40 ⌀ 7mm
90 x 50
7,00 m
20,00 m
1,20
,50
20,00 m

Fig. 5

Fig. 9 is self-explanatory (span about 46 ft.; height from floor to floor about 15 ft. 9 in.). Figs. 10, 11, 12 and 13 show some aspects of the building during its construction.

(*c*) Finally, the most important example of continuity is the Sclayn road bridge in Belgium. It has two spans of 205·72 ft. each, a roadway 23 ft. wide with two footpaths each 5 ft. wide. The structure is a box girder having a total depth of only 6·36 ft. at midspan and 15·58 ft. at the central support. The elevation and cross-section are shown in figs. 14 and 15; a photograph of the finished bridge is given in fig. 18; details of cables are given in fig. 16 and of the prestressing jack in fig. 17. The cables are straight in each span; at their mid-point, above the central support, they are 2·84 ft. higher than at their ends at the end supports. The girder is divided in three

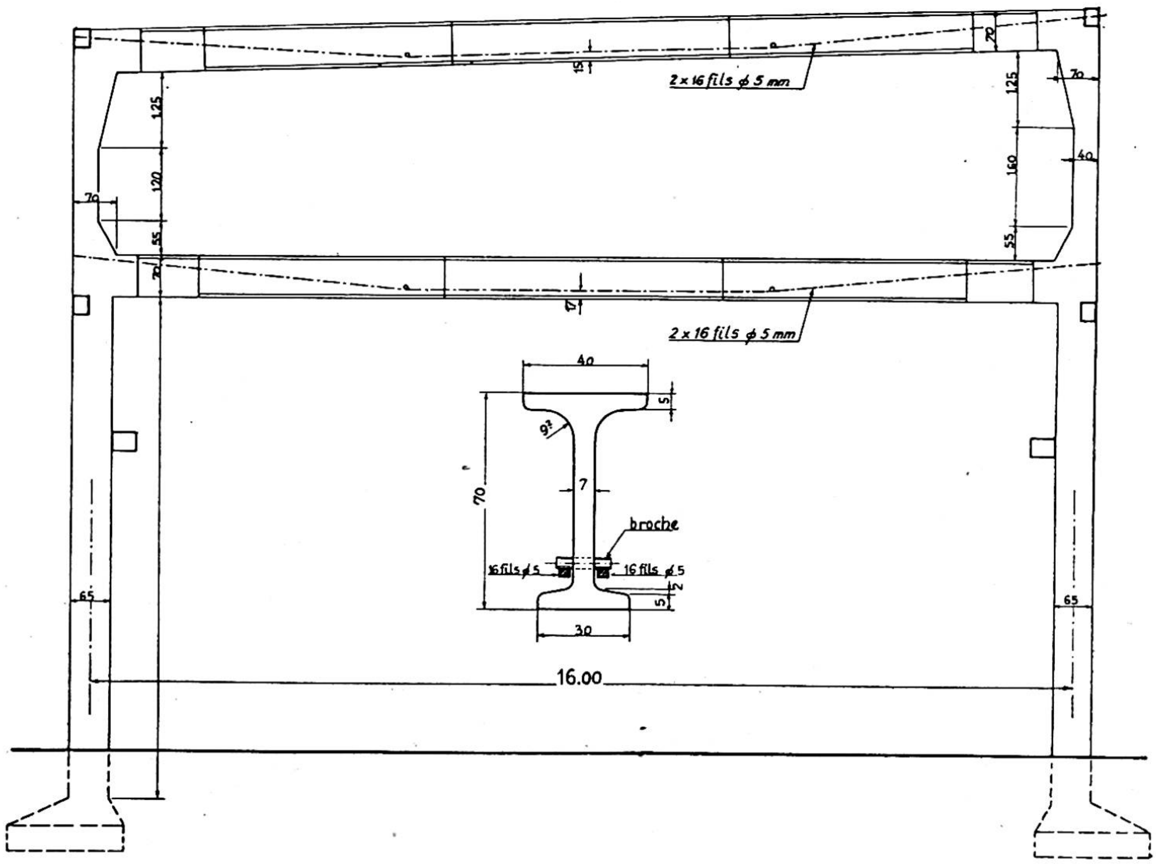


Fig. 6. Factory at Machelen. Depôt—Cross-section

compartments in which the cables are placed. In all 36 cables each of 48 wires of 0·276 in.^{2}, have been provided; they have been prestressed at both ends simultaneously up to 121,000 lb./in.^{2}, which gave initially a total prestressing force of 5,650 metric tons, dropping to about 4,800 metric tons in course of time. The working stress allowed in the concrete is 2,200 lb./in.^{2}

It is worth while pointing out that the secondary bending moment due to the prestressing is in this case initially equal to 67,138,000 lb.-ft., which one should compare with the 128,860,000 lb.-ft. which is the bending moment due to dead and live load at the point above the central support. These figures show that the secondary bending moment is far from negligible in this case. It is helpful above the support, but

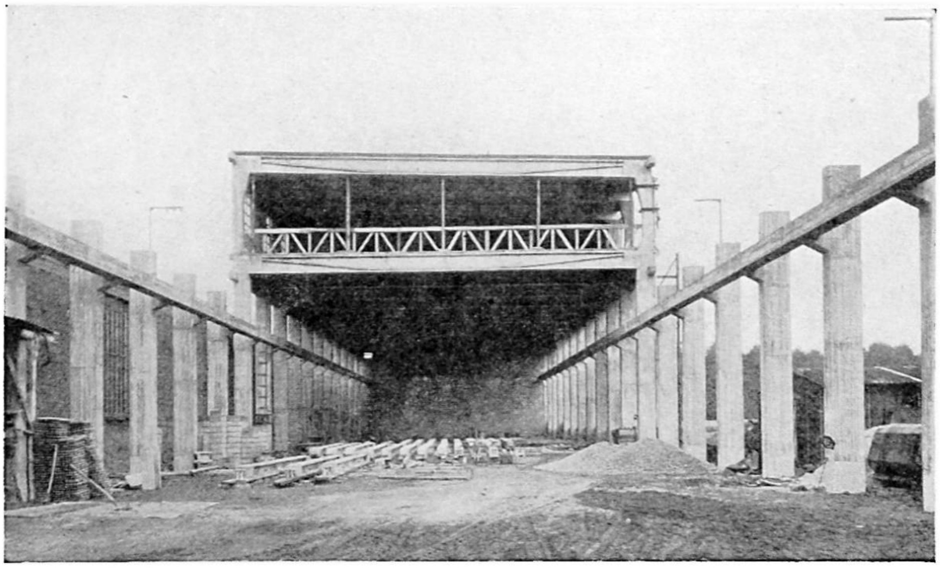


Fig. 7

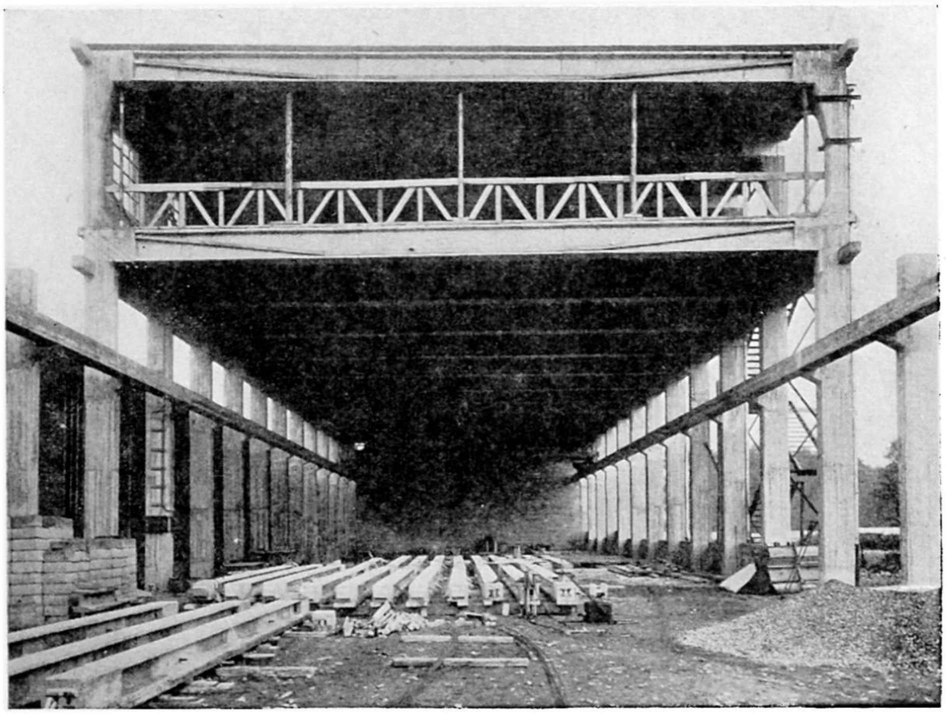


Fig. 8

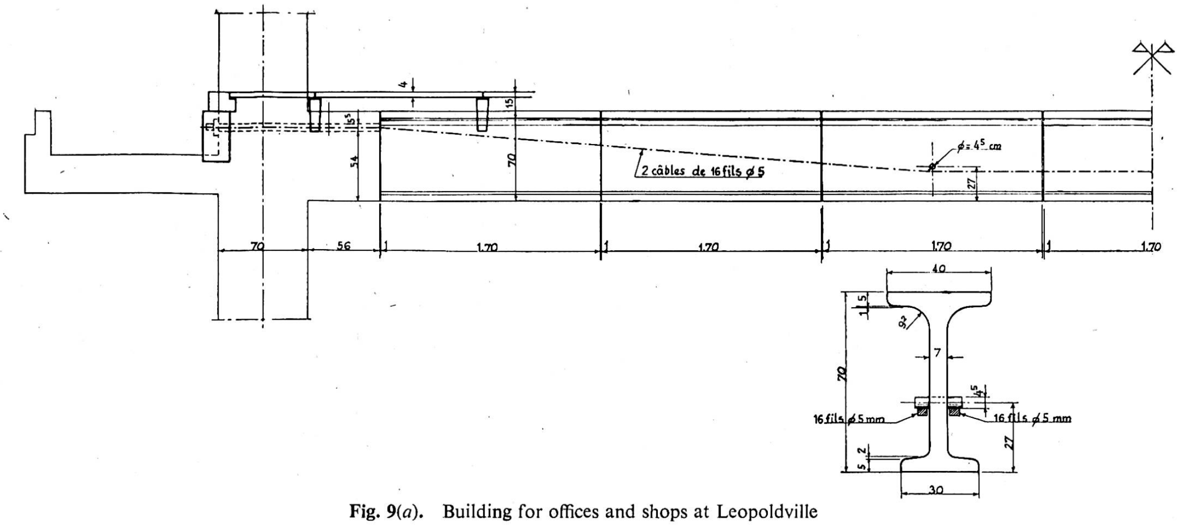


Fig. 9(*a*). Building for offices and shops at Leopoldville

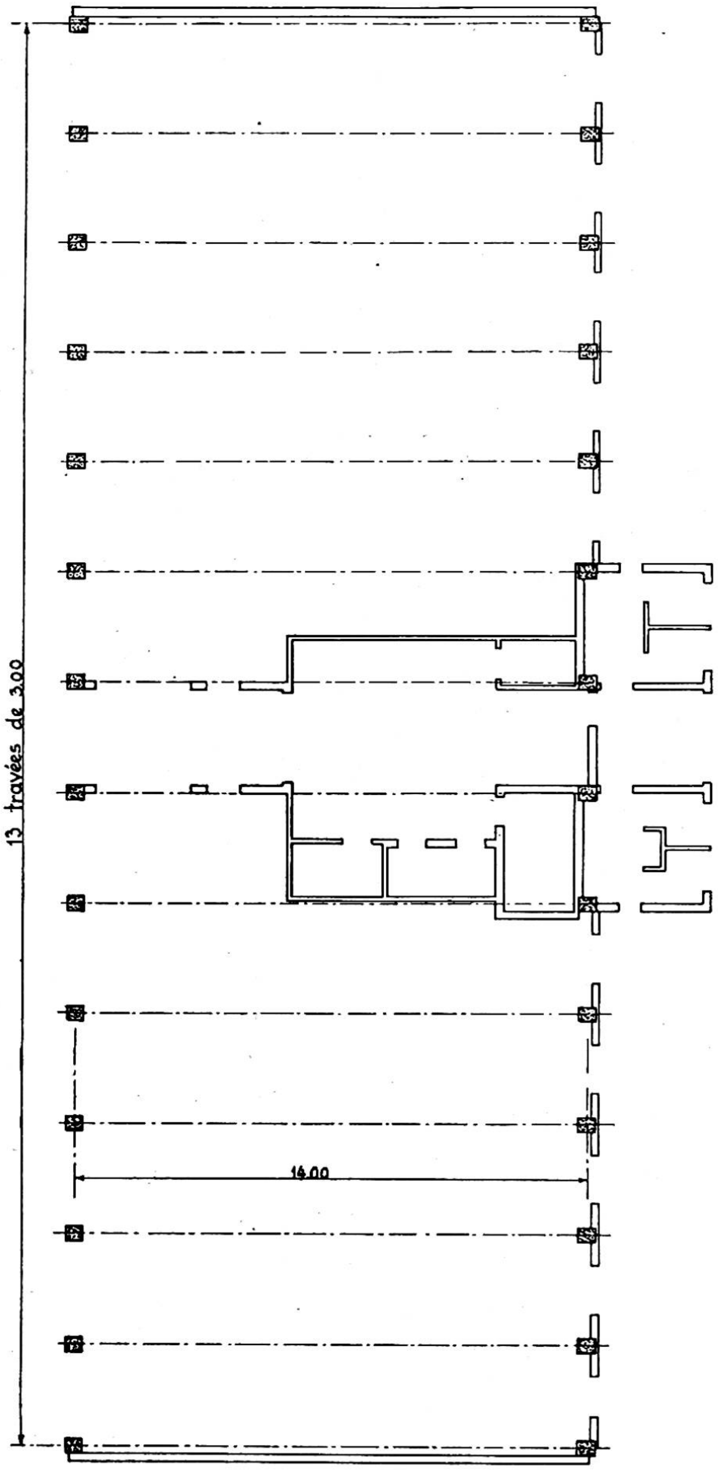


Fig. 9(*b*). Building at Leopoldville—Plan disadvantageous at midspan, when the maximum bending moment due to dead and liveload is 28,500,000 lb.-ft. With another arrangement of the cable the value of the secondary bending moment changes considerably. It is the duty of the designer to find the most economic arrangement.

The Belgian specialists have taken the opportunity of this large bridge to make experiments on the loss of prestress in course of time. Therefore they have provided

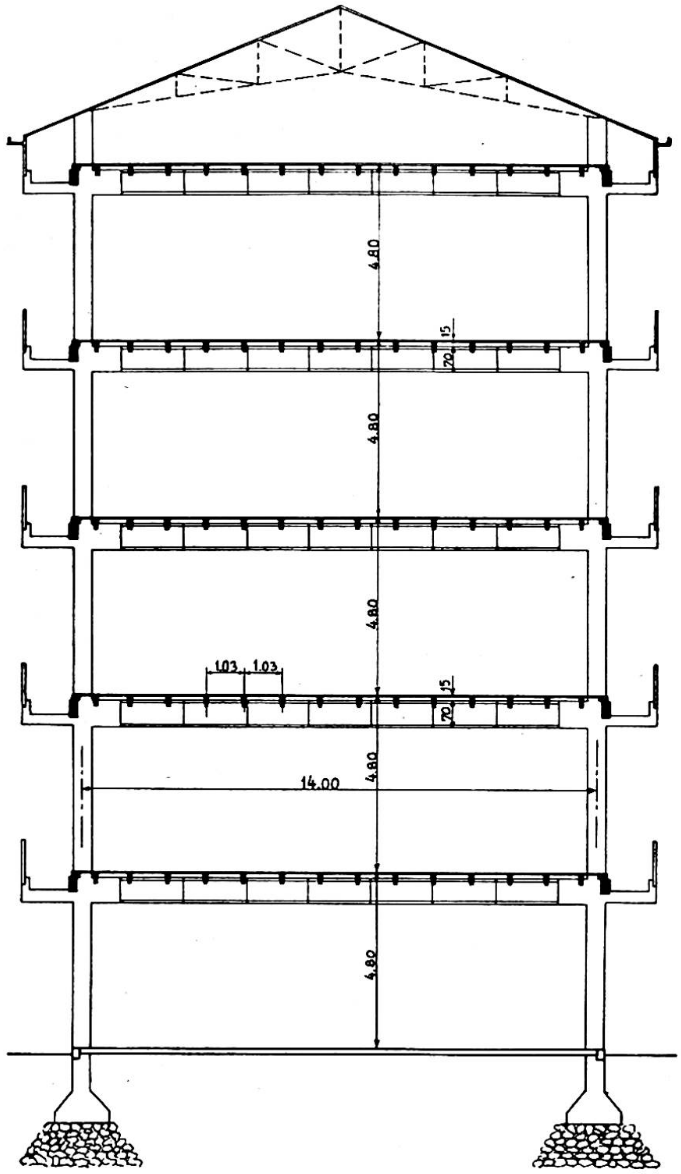


Fig. 9(*c*). Building at Leopoldville—Cross-section

in the box girder two supplementary cables of eight wires each; these cables are not grouted and as their wires remain free, it is possible to measure periodically the variation in stress. Up to the present the measurements made show (after more than two years) that the loss of prestress through all causes is rather smaller than what is generally accepted by designers.

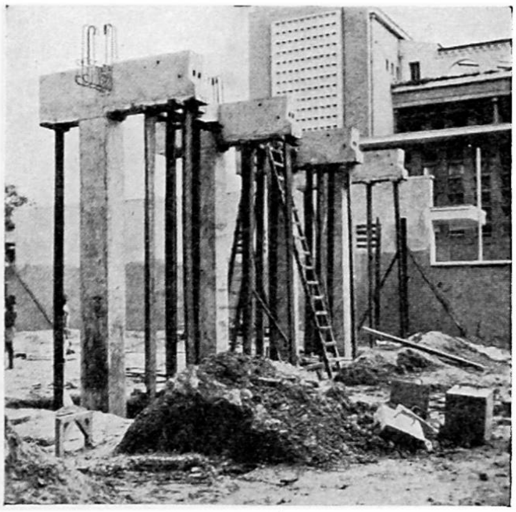


Fig. 10

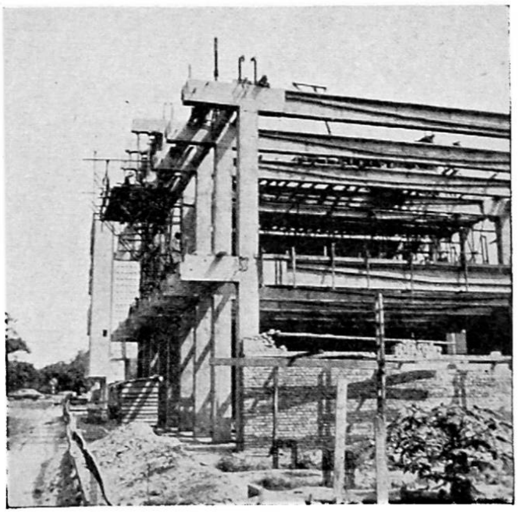


Fig. 11

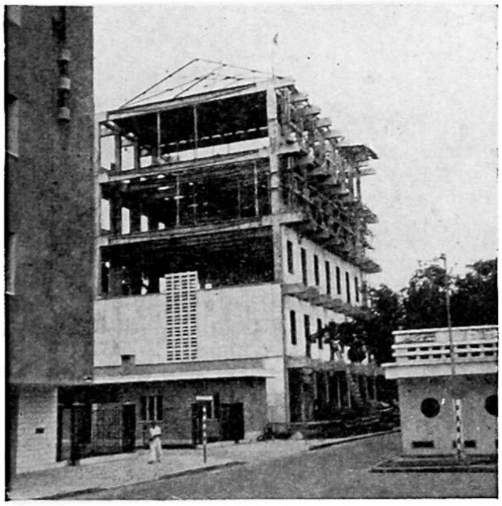


Fig. 12

Fig. 13

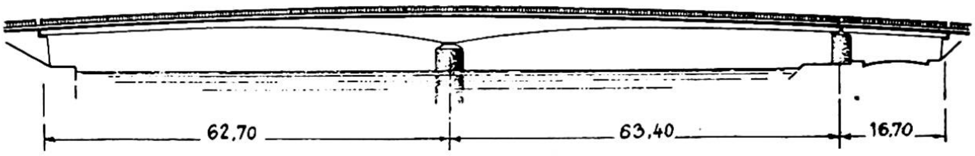


Fig. 14. Bridge over the Meuse at Sclayn in prestressed concrete—Elevation

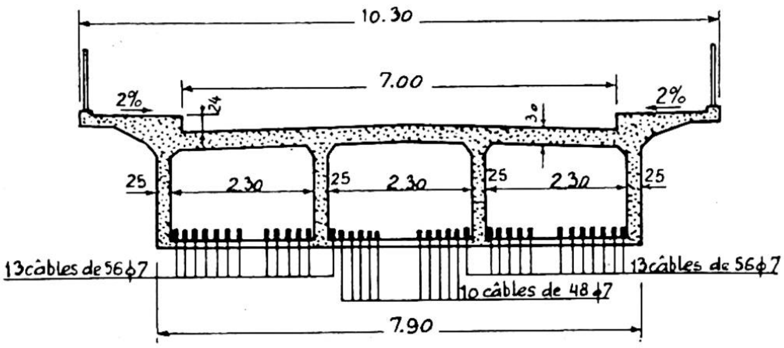


Fig. 15. Bridge over the Meuse at Sclayn in prestressed concrete—Cross-section

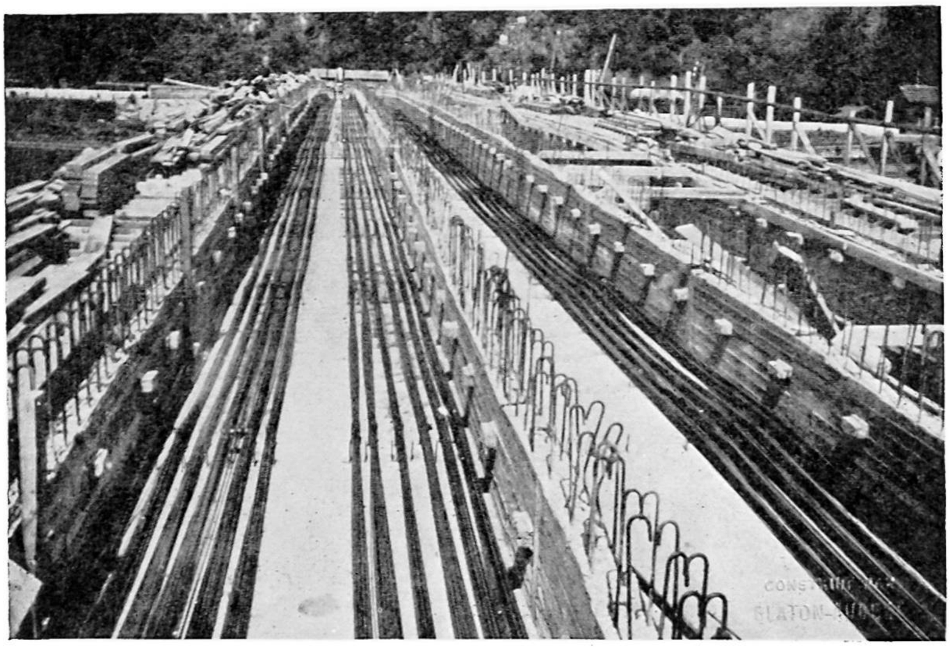


Fig. 16

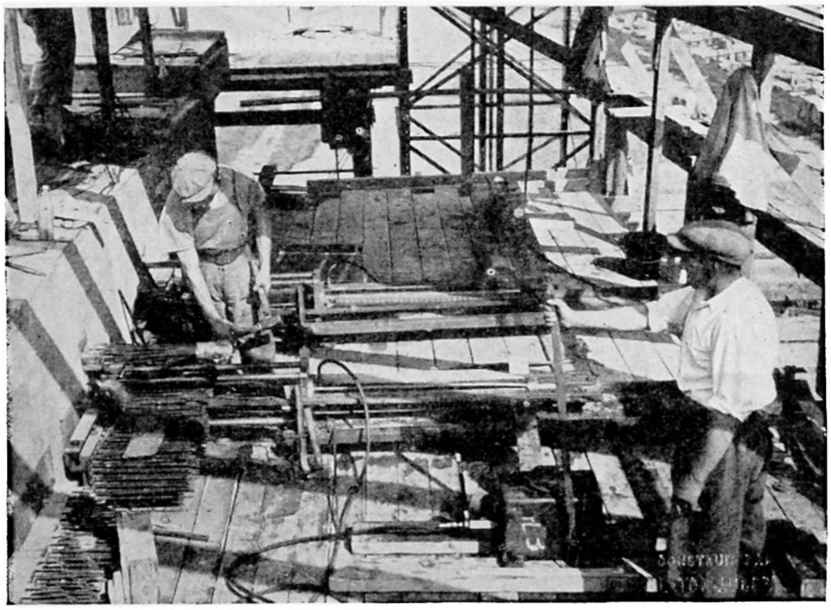


Fig. 17

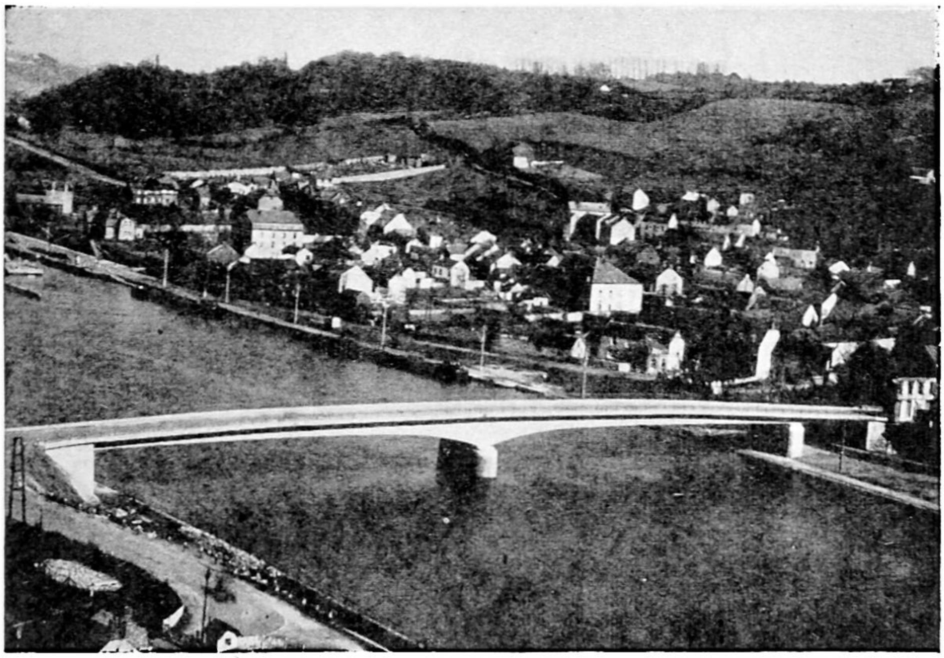


Fig. 18

APPENDIX

RESULTS OF TESTS FOR FRICTION LOSSES

The testing method is shown in fig. 19. A wire (5 or 7 mm. in diameter) is fixed at one end (A) and attached to a jack (D) at the other end (B). The middle of the wire (C) can be deflected by means of a special device; the deflection is called e and the base length l ($l = 13{\cdot}40$ m.).

Strain gauges are attached to the wire at the two places indicated in fig. 19. Details of the cast-iron plate (C) used to cause the deflection are given in fig. 20.

Tests have been made for different values of e by stretching the wire with the jack, and measuring the difference in strain indicated by the two strain gauges for a series of jack loads. The results are summarised in Table I for 5 mm. wires and Table II for 7 mm. wires.

Figs. 21 and 22 show the loss in stress as a function of the angle α for different values of the stress in the wire. The author has checked that the speed with which the wire is stretched to its maximum stress has virtually no effect on the magnitude of the loss due to friction.

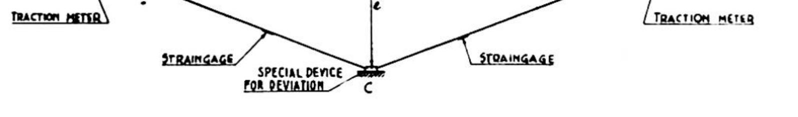
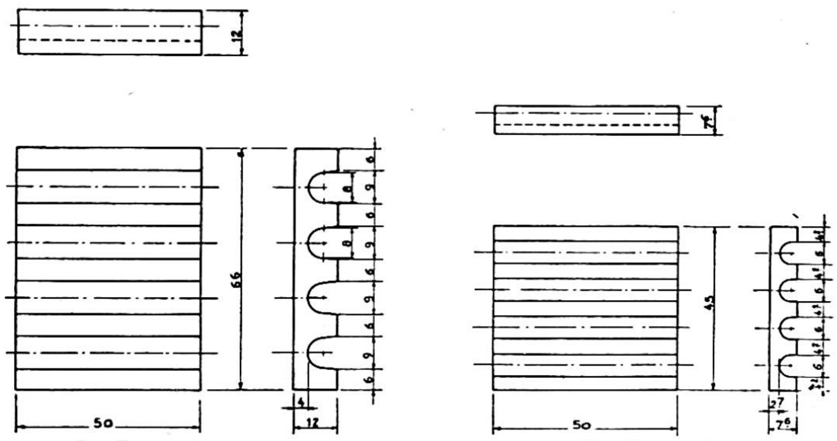


Fig. 19



For 7mm wires Fig. 20 For 5mm wires

TABLE I

Size of wire: 5 mm. diameter

Deviations			Loss of stress due to friction for different stresses in kg./mm.²							
			25		50		75		100	
e (cm.)	α °	′	strain gauge	jacks	strain gauge	jacks	strain gauge	jacks	strain gauge	jacks
0		0	0	0	0	0	0	0	0	0
24	2	3	0·5	—	0·8	—	0·8	—	1·2	—
48	4	5	0·7	—	1·4	—	1·4	—	2·0	—
72	6	10	1·0	—	1·6	—	2·2	—	3·4	—
100	8	30	1·2	—	2·0	—	2·8	—	4·0	—
124	10	30	1·4	1·0	2·4	—	3·4	2·5	5·0	3·1
148	12	30	1·7	1·5	3·0	4·1	4·1	4·6	5·6	5·6
176	14	40	2·0	2·0	4·1	4·1	4·0	4·6	5·9	5·6
200	16	40	2·3	2·5	4·2	4·1	5·6	6·6	8·0	8·1
224	18	30	2·5	3·0	4·0	4·6	6·2	6·6	9·1	8·1

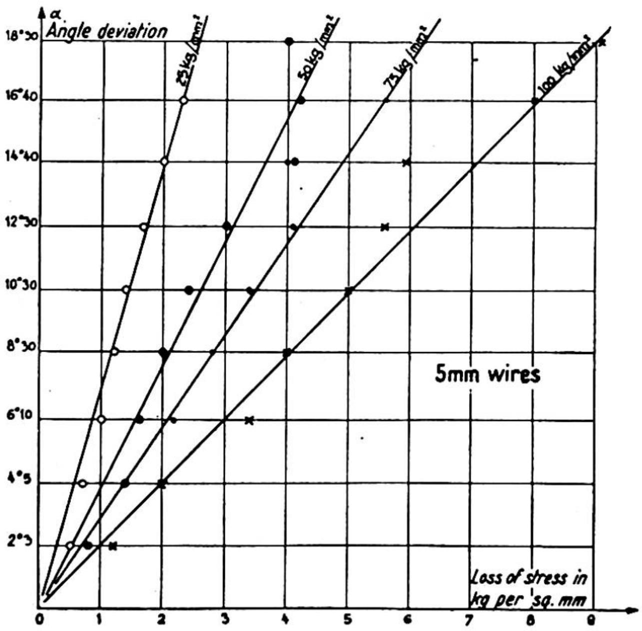


Fig. 21

TABLE II

Size of wire: 7 mm. diameter

Deviations		Loss of stress due to friction for different stresses in kg./mm.²							
		25		50		75		100	
e (cm.)	α ° ′	strain gauge	jacks	strain gauge	jacks	strain gauge	jacks	strain gauge	jacks
0	0	0	0	0	0	0	0	0	0
24	2 3	0·8	—	1·2	—	1·2	0·9	1·5	1·6
48	4 5	0·6	0·6	0·8	1·0	1·2	1·3	2·0	2·1
72	6 10	0·3	0·9	0·5	1·0	1·8	1·9	2·5	2·6
100	8 30	0·3	1·7	1·1	2·1	1·8	2·4	2·7	3·6
124	10 30	1·5	1·3	2·2	2·1	3·2	3·6	4·2	4·4
148	12 30	1·2	2·1	3·0	3·2	4·2	4·9	5·0	6·7
176	14 40	1·3	2·1	3·6	3·9	4·3	5·2	6·2	7·1
200	16 40	2·8	2·1	4·2	3·9	5·2	6·2	7·8	8·2
224	18 30	2·6	2·1	5·1	4·9	7·6	7·0	9·0	9·1

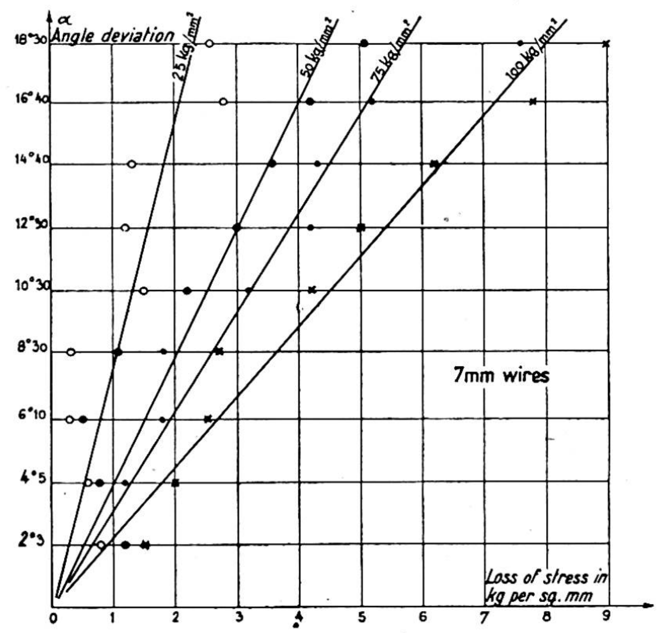


Fig. 22

Summary

The author explains the reasons why it is unavoidable to make continuous statically indeterminate structures in prestressed concrete, and states the theoretical and practical difficulties in connection with this.

Some examples of statically prestressed structures in Belgium are given: these include a two-storey building at Brussels, a four-storey building at Leopoldville, and the Sclayn Bridge across the River Meuse, which is the most important application of continuity made up to the present time in bridge building.

The paper gives some results of measurements of the loss of stress due to friction.

Résumé

L'auteur expose les raisons pour lesquelles il est nécessaire d'associer l'hyperstatisme à la précontrainte; il montre les difficultés corrélatives, tant théoriques que pratiques.

Il cite quelques exemples d'ouvrages hyperstatiques en béton précontraint, réalisés en Belgique: un immeuble à deux étages à Bruxelles, un immeuble à quatre étages à Léopoldville et le pont Sclayn sur la Meuse. Ces exemples constituent les applications actuelles les plus intéressantes de la continuité dans la construction en béton précontraint.

L'auteur termine en reproduisant quelques résultats de mesures concernant les réductions de contraintes dues au frottement.

Zusammenfassung

Der Verfasser erklärt die Gründe, weshalb es unvermeidlich ist, durchlaufende, statisch unbestimmte Konstruktionen in vorgespanntem Beton zu bauen und legt die damit verbundenen theoretischen und praktischen Schwierigkeiten dar.

Einige Beispiele von statisch unbestimmten vorgespannten Bauten in Belgien werden beschrieben: Ein zweistöckiges Gebäude in Brüssel, ein vierstöckiges Gebäude in Leopoldville und die Sclaynbrücke über die Meuse, welche gegenwärtig die wichtigste Anwendung der Kontinuität im Bau vorgespannter Brücken darstellt.

Die Abhandlung enthält einige Ergebnisse von Messungen über den Spannungsverlust infolge Reibung angibt.