IIb: Pre-stressed concrete

Objekttyp: **Group**

Zeitschrift: **IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht**

Band (Jahr): **3 (1948)**

PDF erstellt am: **29.05.2024**

IIb1

Recherches récentes sur le béton précontraint

Neuere Erkenntnisse über vorgespannten Beton

Recent research on prestressed concrete

Dr TECHN. ING. ERICH FRIEDRICH
Professor an der Technischen Hochschule, Graz

1. Einleitung

Im letzten Jahrzehnt wurden hochwertige Stähle im Stahlbetonbau immer mehr verwendet. Dabei müssen wir zwei verschiedene Anwendungsgebiete betrachten. Das erste Gebiet betrifft die Verwendung von hochwertigen Stählen im Stahlbetonbau ohne Vorspannung. In Oesterreich wird in immer grösserem Masse Torstahl verwendet, wobei man erwägt, diesen Stahl mit noch höherer Streckgrenze als bisher herauszubringen. Werden die Stahleinlagen vorgespannt, so gelingt es, Stähle mit noch wesentlich höherer Streckgrenze zu verwenden. Im Nachfolgenden wird über einige Punkte des Bemessungsverfahrens und über hierzu ausgeführte Versuche berichtet.

2. Biegebeanspruchung

Wir sind heute in der Lage im Stahlbetonbau durch Verwendung von Stahl mit hochliegender Streckgrenze und durch Verwendung von Betongüten die Bruchlast von auf Biegung beanspruchten Stahlbetonbalken sehr hoch zu steigern. Der Anwendung dieser hohen Stahlsorten stand jedoch die geringe Riss-Sicherheit dieser Balken im Wege, wodurch unter der zulässigen Belastung, wenn diese als 0,5 bis 0,45 der Bruchlast gewählt wird, bereits klaffende Risse auftreten. Im allgemeinen werden Rissbreiten bis 0,3 mm für den Bestand des Bauwerkes als nicht gefährlich angesehen. Doch ist darauf zu achten, dass wahrscheinlich durch die Haftspannungen Risse nicht Ebenen sondern gekrümmte Flächen sind, so dass Rissbreiten an der Oberfläche, wie Versuche mit Eosin-Einspritzungen zeigten, noch kein Mass für die Beanspruchung des Stahles an der Risstelle angeben. Sie geben nur Vergleichswerte bei gleichen Stahlsorten. Durch die Vorspannung ist man in der Lage nun auch die Last, bei der die ersten Risse auftreten, beliebig hoch zu wählen. Man hat für die Bemessung von vorgespannten Systemen zwei Rechnungsgänge auszuführen :

Der erste Rechnungsgang besteht darin, die Abmessungen des Querschnittes und die Bewehrung, sowie die Höhe der Vorspannung so zu wählen, dass unter den zulässigen Lasten keine Zugspannungen im Beton oder zumindest keine Zugspannungen, die über der Zugfestigkeit des Betons liegen, auftreten. Die Berechnung erfolgt dabei nach Zustand I.

Der zweite Rechnungsgang geht von der Bruchlast aus. Die zulässige Belastung ist so zu wählen, dass bei zwei-facher bis 1,7-facher Sicherheit der Bruch des Balkens eintritt. Erst, wenn der Querschnitt beiden Forderungen genügt, ist er richtig gewählt.

Wenn in einem vorgespannten Träger keine äussere Kraft angreift, ist der Kraftangriff im unteren Kernpunkt oder nahe am unteren Kernpunkt (Lage 0). Unter der zulässigen Belastung wandert die Kraft N in den oberen Kernpunkt (Lage 1). Bei weiterer Steigerung der Last wandert die Kraft aus dem Kern heraus (Lage 2). Die Folge davon ist, dass Zugspannungen auftreten, die zu Rissen führen. Die Berechnung nach Zustand I für solche Belastungsfälle ist nicht mehr zulässig. Der Träger gleicht nun in seiner Wirkungsweise einem Träger, der durch Biegung und Längskraft bei Zustand II beansprucht wird (Bild 1). Die Bruchlasten können daher nach den Regeln für den Zustand II ermittelt werden.

Vorgespannte Systeme sind statisch ebenfalls Stahlbetonträger mit Biegung und Druckkraft. Die Bruchlast kann daher nach den Gesetzen für aussermittig beanspruchte Systeme bestimmt werden.

Die von dem Verfasser ausgeführten Versuche mit Balken mit T-Quer-

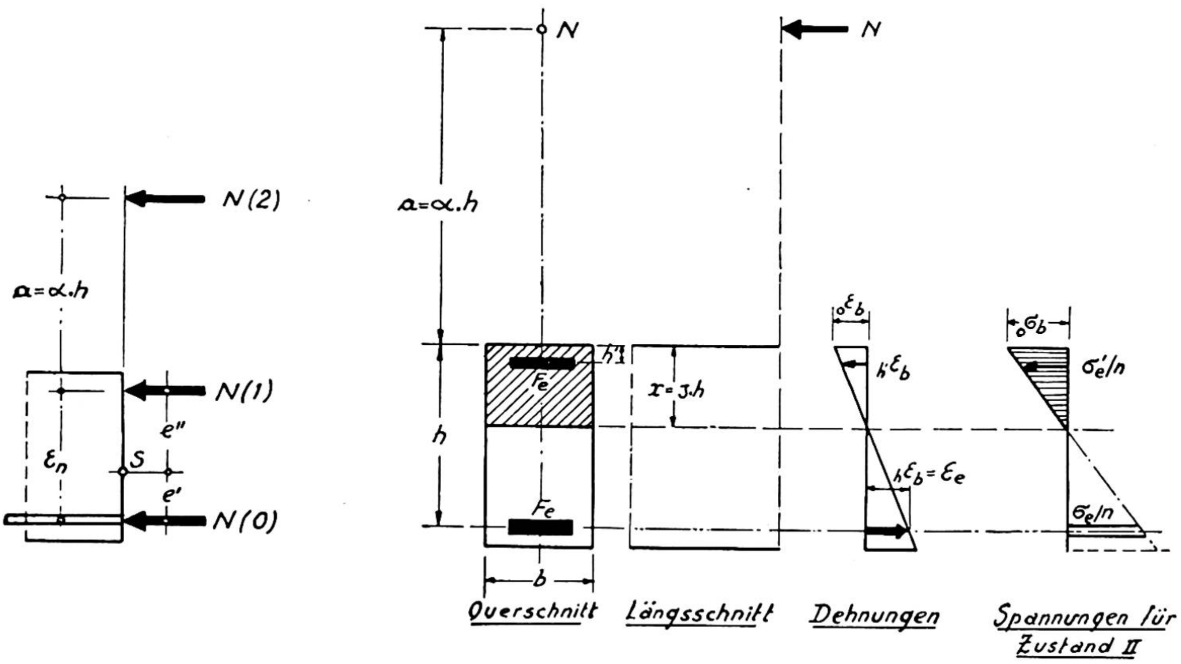
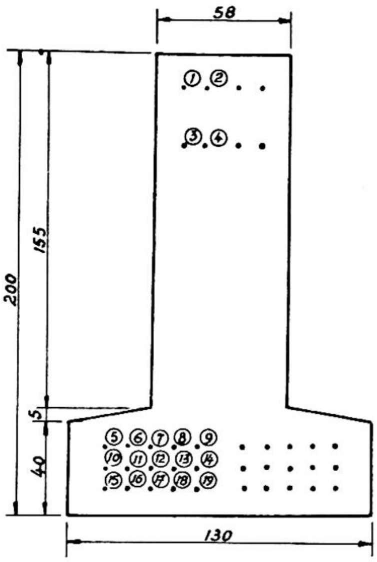


Abb. 1 und 2. Bei Wirkung der Vorspannung allein greift die Druckkraft im unteren Kernpunkt an (0). Bei Wirkung des zulässigen Biegemomentes wandert die Druckkraft in den oberen Kernpunkt (1). Bei weiterer Steigerung des Momentes tritt die Kraft aus dem Querschnitt heraus (2). Der Abstand a der Kraft N von der Druckkante beim Bruch ist ein Mass für die Bruchsicherheit.



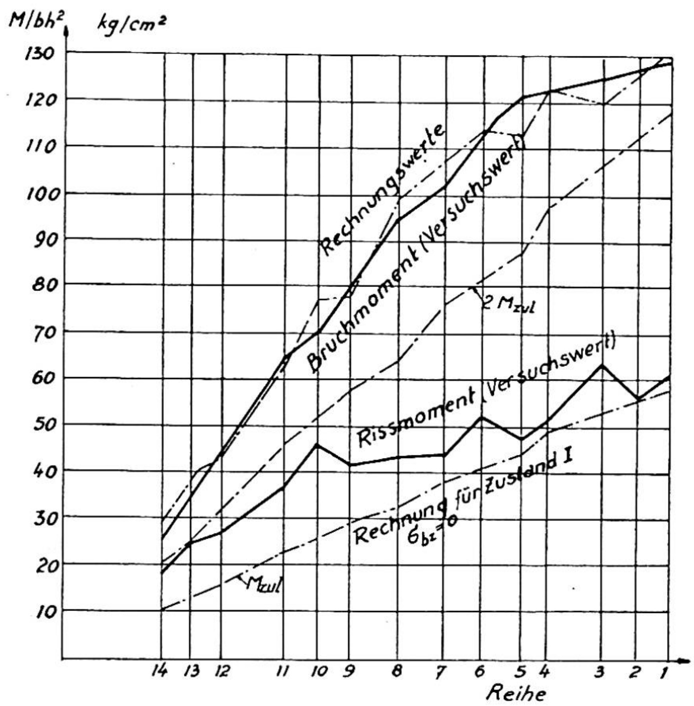


Abb. 3.

schnitt (Bild 3) dienten dazu, diesen Gedankengang zu überprüfen. Die Bewehrung bestand aus Stahlsaiten ⌀ 2 mm. Dabei wurden 14 verschieden bewehrte Balken ausgeführt. Bei der ersten Versuchsreihe waren 38 ⌀ 2 mm eingelegt worden und bei der 14. Reihe waren nur noch 6 ⌀ 2 mm vorhanden. Die Tafel I gibt das Biegungsmoment bei Auftreten der ersten Risse, und das Bruchmoment jeweils als Mittelwert aus drei Versuchen an. Auf dem Bild 3 ist das Ergebnis aufgetragen. Als Abszisse ist dabei die Bewehrungsfläche F_e (cm^2) gewählt worden. Als Ordinate ist das Biegungsmoment bei Auftreten der ersten Risse und beim Bruch angegeben. Diesen Versuchswerten sind jeweils, die nach Zustand I für die Forderung, dass die Betonzugspannung gleich Null ist, und die nach Zustand II ermittelten Werte für das Rissmoment und Bruchmoment gegenübergestellt. Die Gegenüberstellung zeigt, dass das Rechnungsverfahren richtig gewählt ist und im ganzen Bereich der Rissmomentenlinie folgt. Auch die Berechnung der Bruchlast folgt der durch die Versuche bestimmten Linie. Man ersieht, dass man die zulässige Last noch höher wählen kann, indem man etwa eine Zugspannung des Betons mit 1/10 der Würfelfestigkeit zulässt. Selbst auch dann ist die Bruchlast noch stets grösser als die 2-fache zulässige Last. In diesem Fall kann man bei der Bemessung auf den zweiten Rechnungsgang verzichten und noch an Material sparen.

3. Schubbeanspruchung

Einen erheblichen Arbeitsaufwand bei der fabrikmässigen Herstellung von vorgespannten Balken bereitet die zweckmässige Anordnung der Bügel. Da jedoch die Bügel eingelegt werden um die Hauptzugspannungen bei den üblichen Balken aufzunehmen und diese Zugspannungen bei vorgespannten Balken wesentlich kleiner sind, als bei nicht vorgespannten Systemen, ergeben theoretische Ueberlegungen, dass man auf die Bügel über-

Reihe	Anzahl ø mm	Bewehrung Anordnung	Fe cm²
1	38	1, 2 — 3, 4 — 5, 6, 7, 8, 9 — 10, 11, 12, 13, 14 — 15, 16, 17, 18, 19	1,192
2	36	1, 2 — 3, 4 — 5, 6, 7, 8 — 10, 11, 12, 13, 14 — 15, 16, 17, 18, 19	1,130
3	34	1, 2 — 4 — 5, 6, 7, 8, 9 — 10, 11, 12, 13 — 15, 16, 17, 18, 19	1,068
4	30	1, 2 — 4 — 5, 6, 7, 8, 9 — 10, 11, 12, 13 — 15, 16, 18,	0,942
5	28	1, 2 — 4 — 5, 6, 7, 8, 9 — 11, 13 — 15, 16, 17, 18,	0,878
6	26	1 — 3, 4 — 5 — 10, 11, 12, 13, 14 — 15, 16, 17, 19	0,816
7	24	1 — 3, 4 — 10, 11, 12, 13 — 15, 16, 17, 18, 19	0,753
8	20	1 — 3 — 5, 6, 7, 8, 9 — 15, 16, 18,	0,638
9	18	1 — 3 — 10, 13, 14 — 15, 16, 18, 19	0,565
10	16	1, 2 — 10, 13 — 15, 16, 18, 19	0,502
11	14	1 — 4 — 15, 16, 17, 18, 19	0,439
12	10	1 — 15, 16, 17, 18,	0,314
13	8	1 — 17, 18 19	0,251
14	6	1 — 15, 19	0,188

haupt verzichten kann. Man würde dann nur Bügel am Ende der Träger erhalten.

4. Die Berücksichtigung des Kriechens

Um den Spannungsverlust zu berücksichtigen, der durch das Kriechen des Betons entsteht, wurden auf Grund der zahlreichen Versuche folgende Gleichungen aufgestellt (1):

(1) E. Freyssinet, *Une révolution dans les techniques du béton*, Paris, 1936.
R. E. Davis, *Flow of concrete under sustained compression stress* (*Journ. of the Amer. Concrete Inst.*, 1928).
W. H. Glanville, *Studies in reinforced concrete. The creep or flow of concrete under load* (*Technical Paper* 12, London, 1930).
R. E. Davis und H. E. Davis, *Flow of concrete under action of sustained loads* (*Journ. of the Amer. Concrete Inst.*, 1931).
R. Dutron, *Déformations lentes du béton et du béton sous l'action des charges permanentes* (*Ann. Tr. Belg.*, Dez. 1936 bis Febr. 1937).
A. Hummel, *Vom Kriechen und Fliessen des erhärteten Betons und seiner praktischen Bedeutung* (*Zement*, 1935, Heft 50/51).

Schwerpkt-abstand v. oben : mm	Zul. Moment M_{zul} kgm	Rissmoment M_{Riss} Mittelwerte kgm	Bruchmoment M_B Mittelwerte kgm	Bemerkungen
147	1.020	1.051	2.546	Druckzone zerstört
146	976	982	2.463	Druckzone zerstört
151	922	1.101	2 362	Druckzone zerstört
147	850	887	2.328	Druckzone zerstört
145	760	831	2.169	Druckzone zerstört
144	718	910	2.318	Druck- u. Zugzone gleichzeitig zerstört
146	664	769	1.986	Zugzone zerstört
147	558	763	1.905	Zugzone zerstört
149	505	726	1.651	Zugzone zerstört
143	449	813	1.624	Zugzone zerstört
142	395	618	1.368	Zugzone zerstört
153	284	472	991	Zerreissen der Drähte
145	229	440	803	Zerreissen der Drähte
130	172	329	640	Zerreissen der Drähte

TABELLE I.

a) Das Kriechen η_k ist proportional der elastischen Verkürzung

$$\eta_k = k \cdot \varepsilon$$

(Withney'sches Proportionalitätsgesetz);

b) In dieser Gleichung ist k die Kriechfunktion. Für die Kriechfunktion k sind folgende drei Grössen von primärem Einfluss : das Klima, die Kriechschonzeit und die Dauer der Belastung. Das Klima wird durch die relative Luftfeuchtigkeit φ, die Kriechschonzeit durch das Verhältnis α

$$\alpha = \frac{\text{Würfelfestigkeit bei Beginn der Belastung}}{\text{Würfelfestigkeit bei 90 Tagen (Endfestigkeit)}}$$

W. GEHLER, *Hypothesen und Grundlagen für das Schwinden und Kriechen des Betons* (*Bautechnik*, 1938, Heft 30).

F. DISCHINGER, *Untersuchungen über die Knicksicherheit, die elastische Verformung und das Kriechen des Betons bei Bogenbrücken* (*Bauingenieur*, 1937, Heft 33 bis 40).

F. DISCHINGER, *Elastische und plastische Verformungen der Eisenbetontragwerke und insbesondere der Bogenbrücken* (*Bauingenieur*, 1939, Heft 5/6 u.f.).

Lagerungsart	$a = \frac{W_{bt}}{W_{b90}}$ (¹)	Belastungsdauer		
		5,6 (1 Woche)	14 (2 Wochen)	36 (1 Monat)
Aeusserst feuchte Luft (z.B. Bauten am Meer) I. 80 % bis 100 % relat. Feuchigk. Mittelw. $\varphi_m = 90$ %	1,00 0,90 0,80 0,71	0,20 0,25 0,315 0,40	0,25 0,315 0,40 0,50	0,315 0,40 0,50 0,63
Sehr feuchte Luft (z.B. Bäder) II. 63 % bis 80 % $\varphi_m = 71$ %	1,00 0,90 0,80 0,71	0,25 0,315 0,40 0,50	0,315 0,40 0,50 0,63	0,40 0,50 0,63 0,80
Feuchte Luft (z.B. Flusstäler) III. 50 % bis 63 % $\varphi_m = 56$ %	1,00 0,90 0,80 0,71	0,315 0,40 0,50 0,63	0,40 0,50 0,63 0,80	0,50 0,63 0,80 1,00
Trockene Luft (z.B. Bauten im Freien) IV. 40 % bis 50 % $\varphi_m = 45$ %	1,00 0,90 0,80 0,71	0,40 0,50 0,63 0,80	0,50 0,63 0,80 1,00	0,63 0,80 1,00 1,25
Sehr trockene Luft (z. B. trockene Innenräume) V. 32 % bis 40 % $\varphi_m = 36$ %	1,00 0,90 0,80 0,71	0,50 0,63 0,80 1,00	0,63 0,80 1,00 1,25	0,80 1,00 1,25 1,60

und die Zeitdauer t in Tagen ausgedrückt. Aus den Versuchen kann man die Beziehung ableiten :

$$K = 0{,}71 \cdot \frac{\sqrt[4]{\frac{t}{1.400}}}{\alpha^2 \cdot \varphi} \quad \text{für } t \leqslant 1.400 \text{ Tage (rund 4 Jahre)}$$

$$K = K_{max} = \frac{0{,}71}{\alpha^2 \cdot \varphi} \quad \text{für } t = 1.400 \text{ Tage (rund 4 Jahre)} .$$

Für die praktische Rechnung dient die aufgeführte Tafel II, wobei die Zahlenwerte aus den obigen Gleichungen berechnet sind. Damit ist das Kriechmass auf eine elastische Formänderung zurückgeführt, so dass die Berücksichtigung in der Rechnung keine Schwierigkeit bereitet.

5. Anwendungsbeispiele

Die Hauptanwendungsgebiete vorgespannter Betonbauteile sind in drei Gruppen einzuteilen :

a) Massenherstellung : Träger für den Wohnungsbau, Schwellen für Eisenbahnschienen und ähnl. Bei diesen Trägern liegt der Vorteil der Vor-

in Tagen			
90 (1/4 Jahr)	224 (7 Monate)	560 (1 1/2 Jahre)	1.400 (2) (4 Jahre)
0,40	0,50	0,63	0,80
0,50	0,63	0,80	1,00
0,63	0,80	1,00	1,25
0,80	1,00	1,25	1,60
0,50	0,63	0,80	1,00
0,63	0,80	1,00	1,25
0,80	1,00	1,25	1,60
1,00	1,25	1,60	2,00
0,63	0,80	1,00	1,25
0,80	1,00	1,25	1,60
1,00	1,25	1,60	2,00
1,25	1,60	2,00	2,50
0,80	1,00	1,25	1,60
1,00	1,25	1,60	2,00
1,25	1,60	2,00	2,50
1,60	2,00	2,50	3,15
1,00	1,25	1,60	2,00
1,25	1,60	2,00	2,50
1,60	2,00	2,50	3,15
2,00	2,50	3,15	4,00

(1) W_{bt} = Würfelfestigkeit zur Zeit des Aufbringens der Belastung auf den Beton ;
W_{b90} = Würfelfestigkeit im Alter von 90 Tagen (Endfestigkeit)

$$\eta_t = k \cdot \varepsilon$$

$$k = 0{,}71 \frac{\sqrt[4]{\frac{t}{1.400}}}{\alpha^3 \varphi}$$

(2) Endkriechmass

TABELLE II : *Tafel zur bestimmung des Kriechmasses* $K = \frac{\eta_t}{\varepsilon}$

spannung insbesonders im kleinen Gewicht der Träger. Die Bauteile müssen so geformt werden, dass die Herstellung in Strassen von rd. 100 m Länge möglich ist, wobei auf die leichte Entformung Rücksicht genommen werden muss. Die Herstellung von Eisenbahnschwellen aus Stahlbeton ist ein Gebiet, das nicht nur in Oesterreich sondern auch in der Schweiz eingehend bearbeitet wird ;

b) Für Bauteile mit grosser Spannweite. Für viele Bauteile, wo der übliche Stahlbeton praktisch nicht mehr verwendet werden kann, liegt ebenfalls ein Anwendungsgebiet des vorgespannten Betons. Hier handelt es sich meist um Einzelanfertigungen. Die Geräte, die zur Aufbringung der Vorspannung auf der Baustelle dienen, sind zum grossen Teil entwickelt ;

c) In vielen Fällen sind jedoch auch Bauteile zu konstruieren, bei denen es nicht darauf ankommt, eine hohe Tragfähigkeit zu erzielen, sondern viel mehr darauf, grosse Formänderungen ohne Risse aufzunehmen. Eine solche Anwendung ist z.B. die Anordnung einer vorgespannten Dichtungswand in Speicherdämmen (Bild 4). Die Aufgabe dieser Dichtungswand besteht darin, den Damm abzudichten. Eine Stahlbetonwand üblicher Konstruktion ist nicht geeignet, da sie durch die im Inneren der Dämme auftretenden Bewegungen bricht. Durch die Vorspannung der Wand ist sie in der Lage, die grossen Formänderungen aufzunehmen.

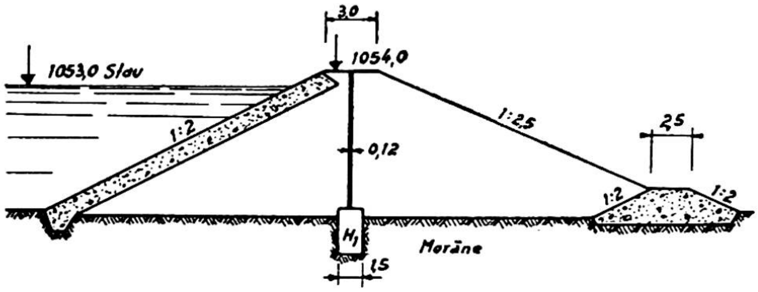


Abb. 4.

Sehr vorteilhaft lassen sich aus vorgespannten Beton Schalungsplatten herstellen. Die Schalungskörper werden hinterfüllt und verbleiben als verlorene Schalung am Betonkörper. Sowohl beim Bau von tunnelförmigen Körpern, wie auch beim Bau von Massivbauwerken, wie z.B. bei Staumauern ist diese Methode sehr vorteilhaft. Auch im Brückenbau ergeben sich hier für die Vorspannung neue Anwendungsgebiete.

Résumé

Pour les constructions précontraintes, il faut considérer deux cas de charges pour la détermination des charges admissibles. Le premier cas est la charge de fissuration; elle dépend principalement de la valeur de la précontrainte. Le deuxième cas est la charge de rupture dépendant des qualités des matériaux.

Ce mémoire relate des essais établissant une liaison entre ces deux cas et indique les procédés de mesure.

Zusammenfassung

Bei vorgespannten Systemen sind für die Festlegung der zulässigen Belastung zwei verschiedene Laststufen zu berücksichtigen. Die erste Laststufe ist die Risslast. Sie hängt wesentlich von der Höhe der Vorspannkraft ab. Die zweite Laststufe ist die Bruchlast, die von den Baustoff-Festigkeiten abhängt.

Im Bericht werden Versuche angegeben, die den Zusammenhang der beiden Laststufen darlegen und Bemessungsverfahren angezeigt.

Summary

With prestressed systems two varying load-stages have to be taken into consideration when laying down a permissible load. The first stage is the projection load. This depend mainly on the extent of the prestressing. The second stage is the rupture load, which depends upon the strength of the building material.

In the report mention will be made of tests which show the connection between the two load stages and measuring processes.

IIb2

Travaux de recherches et de fabrication d'éléments de béton précontraint, réalisés à la « Field Test Unit, Ministry of Works » à Londres

Forschungsarbeiten und Herstellung von vorgespannten Eisenbeton-Fertigteilen, durchgeführt von der « Field Test Unit, Ministry of Works »

Research work and test production of prestressed concrete units at the Field Test Unit, Ministry of Works, London

KURT BILLIG
Chartered Civil Engineer, London

Anticipating the increasing difficulties in the supply of timber and steel — and of foreign exchange, 1946, the Chief Scientific Adviser to the Ministry of Works gave instructions to investigate the use of prestressed concrete as a substitute for timber and steel for transmission poles, telegraph poles, structural beams, floor units, etc. Being the Consultant to the Ministry on prestressed concrete the Author submitted the program and supervised its execution.

The specification of the materials employed in all experiments and test production were :—

Steel : 140-150 ton tensile; 2 per cent proof stress 110-120 ton per sq. in. Twin-twisted strands of compressor wires of S. W. G. 12 and 11. No creep under fatigue test between 75 and 85 ton per sq. in. Wire delivered in coils shall straighten when uncoiled. Concrete : cube crushing strength, age 7 days, 6 000 lb per sq. in; age 28 days, 9 000 lb per sq. in.

1. The initial investigations dealt with the determination of the embedding length required to develop the full tensile strength of the wire strands in the concrete. Result : The anchorage length of twin strands, at their ultimate tensile strength, amounts to less than 120 dia. of the single wires.

2. Investigations on the bond of wires in castings of rope capping metal for the purpose of gripping and stretching. Composition of metal : 80 % lead + 15 % antimony + 5 % tin. Preliminary tinning of the wires to

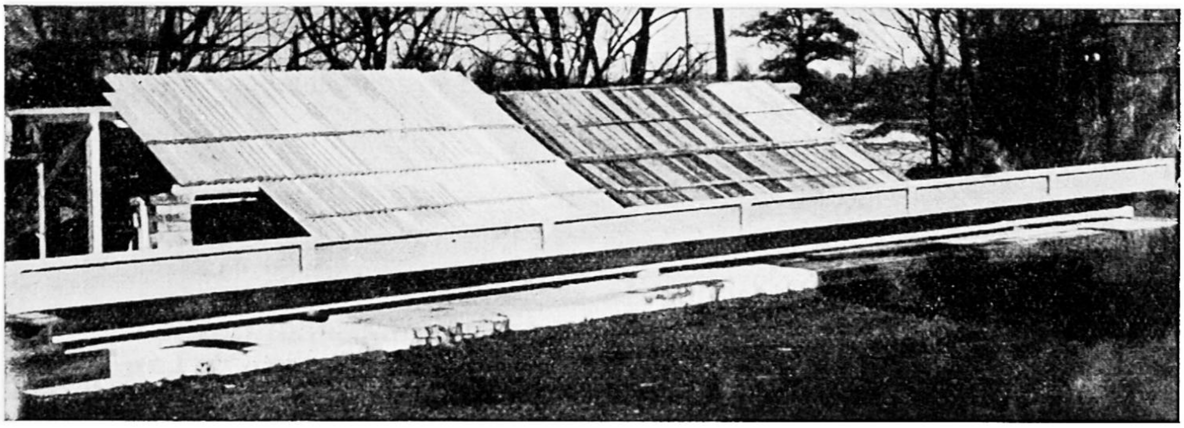


Fig. 1. Prestressed concrete joist. Depth-span ratio 1:45.

improve bond. Result : The anchorage length of twin strands amounts to approx. 60 dia. of the single wires.

3. Investigations on mechanical devices to grip compressor wires. Type suitable to anchor single wires : tapered pins fitting into tapered holes of steel plates with grooves to receive the wires; pins and bushings of holes to be of hardened steel. Type suitable to anchor twin strands : as above, but pins one side flattened and cross riffled, or split with central thread gripping.

4. Test production of prestressed concrete I-Joists No. 12, 45 ft long each. See fig. 1. Cross section 6″ × 12″, 2″ flanges, 1 1/4″ web, dead weight 35 lb per lin. ft. Compressor wires 28 twintwisted S. W. G. 12, weight of steel 1.62 lb per lin. ft evenly distributed over the whole cross section, ten in each flange, eight in the web. Initial pre-tension 90 ton per sq. in, effective 75 ton per sq. in. Effective uniform precompression of the concrete one ton per sq. in. There is no preliminary bending in these units. The units have equal resistance to positive and negative bending, positive or negative shear, left and right hand torsion. Bigger units of this type are intended to replace R. S. J's in structural engineering. Profile No. 12 as above is designed to carry a floor load of 1 cwt per sq. ft (in

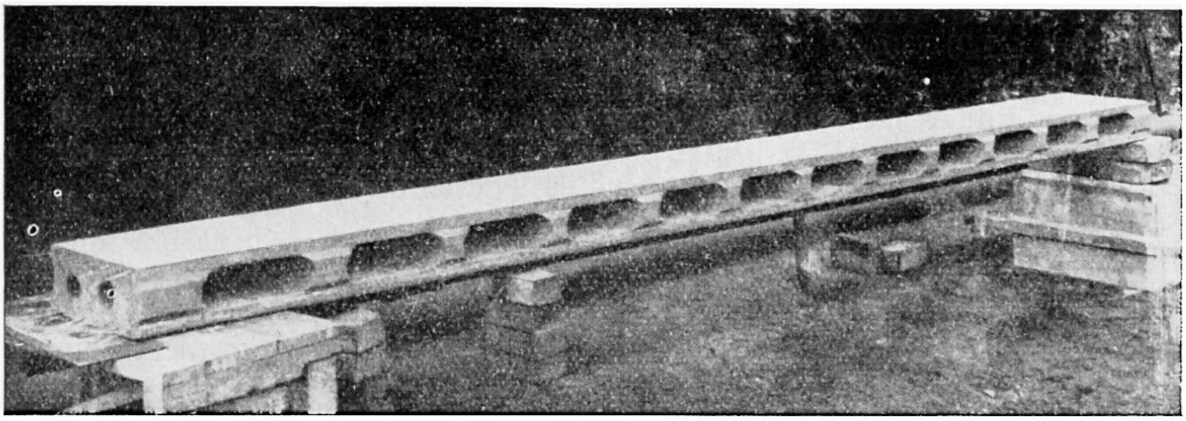


Fig. 2. Prestressed concrete hollow floor unit.

public buildings) over a clear span of 30′0″ and with cantilevers of 7′6″ at each end to carry the outer walls — depth of building 45 ft — the prestressed concrete beams being spaced at 18″ c/c.

5. Differential shear stresses. During the production of one of these units 2 wires positioned in the web slipped when all wires were stretched together. The two wires were not re-stretched but embedded without prestress. After release, a longitudinal crack appeared in the beam between the two slipped wires, 3/4″ wide at the end, and 6′ 9″ long, progressing towards midspan.

Conclusion : With so high prestresses, it is essential to keep the precompression uniform over the whole cross section.

6. Test production of prestressed concrete hollow floor units, 21 ft long. See fig. 2 Cross section 5″ $\times$ 12″ with central cavity 3 1/2″ $\times$ 10″, 3/4″ slabs top and bottom, each containing 10 compressor wires single S. W. G. 16, parts of web omitted. Dead weight 24 lb per sq. ft, weight of steel 20 lb per sq. yd of floor.

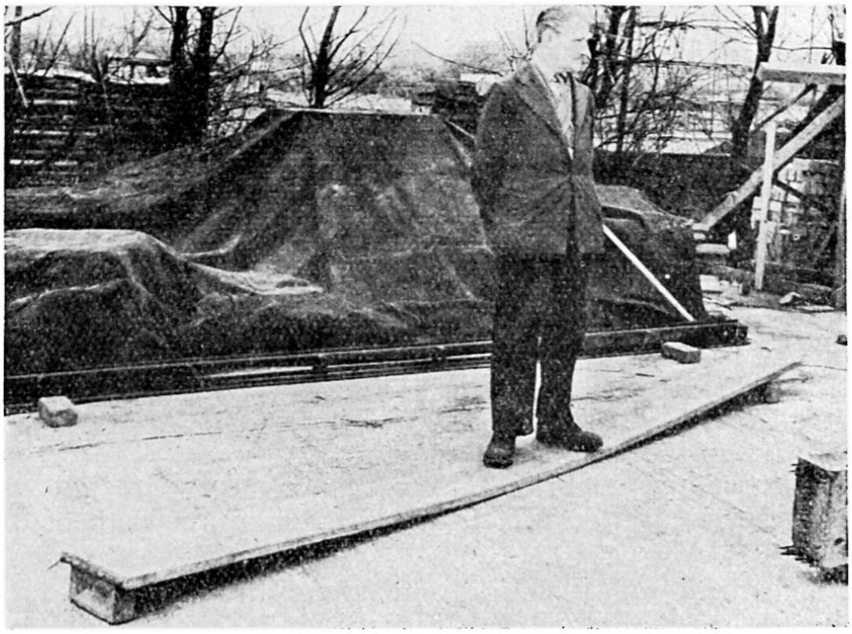


Fig. 3. Prestressed concrete plank 1 1/4″ thick, 12 ft span.

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These floor beams are designed to span 20 ft from the front wall to the back wall of standard houses. The spine wall may be positioned anywhere from 7 ft to 13 ft distance from the front wall. The load bearing spine wall in the ground floor may be erected after the prestressed concrete units forming the first floor have been laid. To take the live loads the prestressed concrete units work as a continuous floor over three supports.

For loading tests and fire tests see seperate Reports issued by the Building Research Station, Watford, and Fire Testing Station, Elstree.

7. Test production of prestressed concrete planks, 1 1/4″ tick, 12″ wide, 12′9″ long. The elasticity of the planks is shewn by fig. 3. The concrete is precompressed to 400 lb per sq. in only.

Prestressed concrete planks, compressed to one ton per sq. in, 3/4″ thick, 9″ wide, have been designed to be used as scaffold boards, 5 ft long, laid on longitudinal supports.

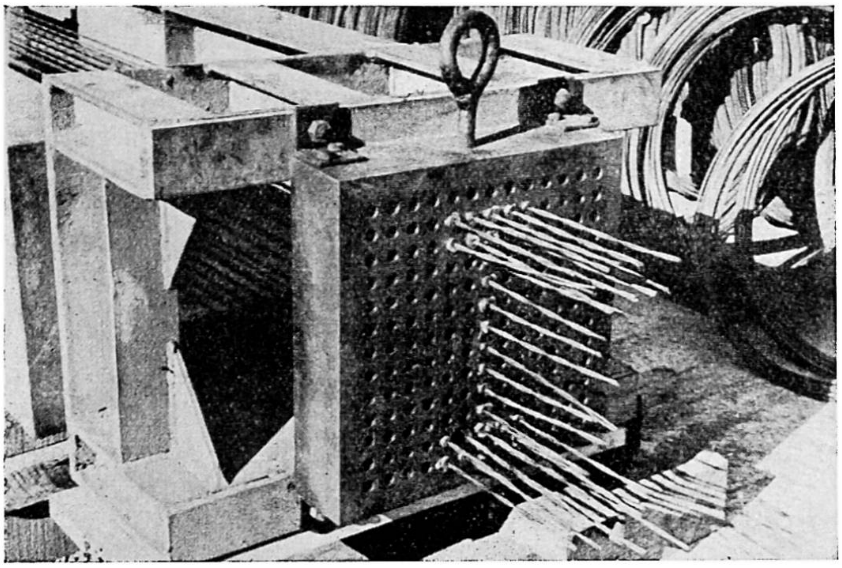

Fig. 4. Universal cross head and grid plate.

To illustrate the elasticity of prestressed concrete, similar planks, 2″ thick, have been designed to be used as diving boards.

8. Test production of Compound Reinforcement. Compound reinforcement is a member consisting of compressed concrete pipes containing stretched steel wires, the internal reactions between both materials being transferred by cement grout bonded to both.

To produce a compound reinforcement precast pipes are laid in one line, and the wires are threaded through. The wires are then stretched and the pipes are compressed by means of end anchorages. Grout is filled into the central hole and establishes solid connection between the wires and the pipes. The end fittings are removed after the grout has hardened. Example : External diameter of concrete pipes 4″; thickness of wall 1″. Compressor wires : 24 wires S. W. G. 10. This unit is equivalent to a reinforcement of 3 ϕ 1 1/8″ mild steel bars, the cross-sectional area of which is approx. 10 times greater than that of the compressor wires 24 ϕ S. W. G. 10.

Although this Compound Reinforcement is a prestressed concrete unit, the concrete structure in which it is used is, for all intents and purposes, an ordinary reinforced concrete structure, and the builder is not concerned with any prestressing apparatus.

9. Equipment developed during experimental production.

a) Gripping devices.

Concrete blocks cast around ends of wires, and wires stretched in groups. See paragraph 1.

Castings of rope capping metal round ends of wires, and wires stretched in groups. See paragraph 2.

Mechanical grips. Wires stretched singly or in groups. See paragraph 3.

b) Stretching devices.

3-ton hydraulic jack to stretch single compressor wires or strands. Universal crosshead and grid plate, see fig. 4, which can be employed for units with up to 169 wires or wire strands. Maintenance of stretching force by steel struts.

c) Gauges.

Statimeter gauges of up to 100 ton.

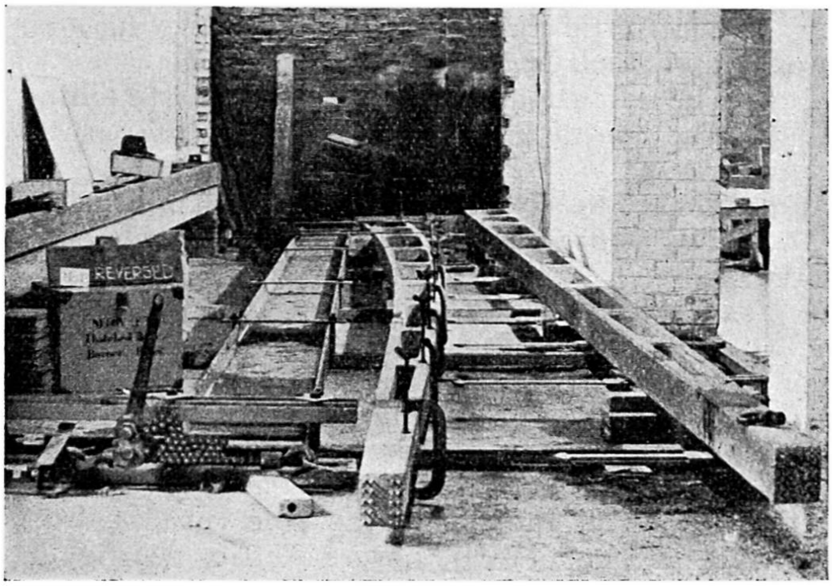

Fig. 5. Prestressed concrete pole under test load 2 100 lb.

10. Test production of prestressed concrete transmission poles.

Specification : Class B, British Standard 607-1946. O/a length 36 ft. Planting depth 6 ft. Ultimate load, 2 ft from top, 1 250 lb in a direction transverse to the transmission line.

Design A. See fig. 6.

A Vierendeel frame with two legs 2 1/8″ thick, the cross section tapering from 17″ × 7.2″ to 5″ × 6″. The diaphragms spaced from 4′3″ to 3′0″ centres, are not prestressed; neither is the 2″ web extending over the planting depth. Each leg is precompressed by 10 twin wires S. W. G. 11; the diaphragms and web are reinforced by high tensile steel.

Design B. See fig 6.

A full web I-section tapering from 13.8″ × 6.2″ to 11″ × 5″ with flanges 2 1/8″ thick and the web 1 1/4″ thick above ground level and 1 3/4″ thick below ground. The whole cross section is precompressed by 28 twin wires S. W. G. 11, ten in each flange, eight in the web.

Test production of 10 poles, 5 of each type.

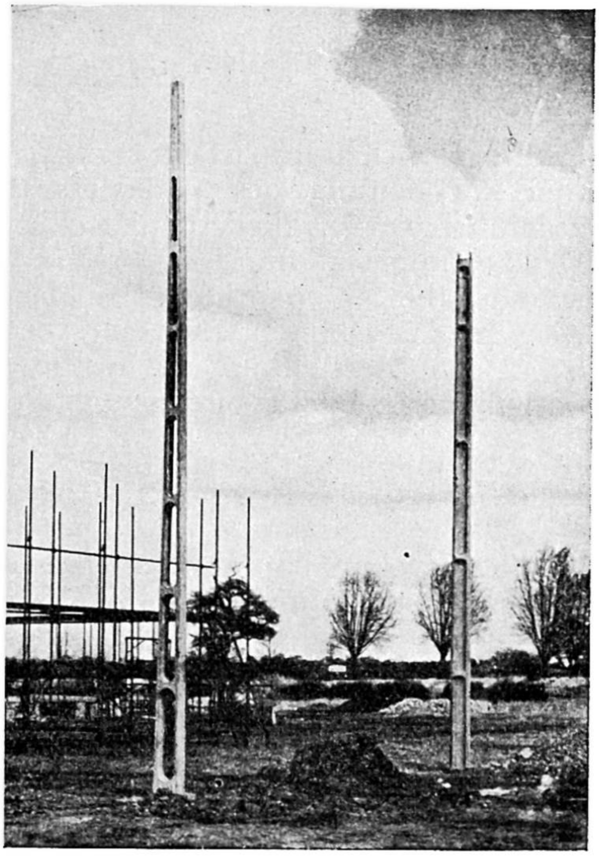

Fig. 6. Two types of prestressed concrete poles.

Specification of concrete and steel, and prestresses, as before.

To control the stresses set up the following alterations in length were measured in each specimen :—

a) The extension of the steel during stretching;

b) The contraction of the ram with the piston running back during release;

c) The contraction of the concrete pole during release;

d) The contraction of the concrete pole after release, while it is stored.

For loading tests of all ten specimens see separate Report issued by the Structural Section of the Ministry of Works. The most remarkable result of these tests is the small divergence of the characteristic load values in all ten poles.

The first cracks appeared under loads between 1 200 and 1 400 lb.

Failure occurred under loads between 2 000 and 2 250 lb.

Deflection at the top of the pole, just before failure, reached up to 40 in, 97 percent of which were elastic. See fig. 5.

Résumé

Les travaux pour le développement des éléments en béton précontraint, réalisés sous la direction de l'auteur pour le compte du Ministère des Travaux, Londres, au cours des années 1947-1948 se subdivisent en deux parties :

1. Recherches des diverses méthodes pour la fixation, l'ancrage et la tension des fils ainsi que l'adhérence du béton.

2. La fabrication et l'essai des éléments en béton précontraint, notamment des poutres, dalles et poteaux.

Zusammenfassung

Die Entwicklungsarbeit an vorgespannten Betonfertigfabrikaten, die unter der Leitung des Verfassers für das Bauministerium in London ausgeführt wurde, gliederte sich wie folgt :

(1) Untersuchung von verschiedenen Methoden für die mechanische Fassung, die Verankerung und das Spannen der Drähte; die Haftung zwischen den Drähten und dem vorgespannten Beton, etc.

(2) Die Herstellung und Prüfung von vorgespannten Betoneinheiten, wie I-Trägern, Deckenplatten und Freileitungsmasten.

Summary

The development work on prestressed concrete carried out under the Author for the Ministry of Works, London, 1947-48, consisted of two parts :—

(1) Research on various methods for the gripping, fixing and stretching of compressor wires, the bond between the wires and the compressed concrete in the prestressed unit etc.

(2) The experimental manufacture and test loading of prestressed concrete units, such as I-joists, floor slabs and transmission masts.

IIb3

Le comportement du béton précontraint après fissuration
(Données pour le projet d'un tel ouvrage)

Das Verhalten von vorgespanntem Beton bei Rissebildung
(Folgerungen für den Entwurf)

The behaviour of prestressed concrete at cracking
(Conclusions for the design)

PAUL WILLIAM ABELES
D. Sc. (Vienna), M. I. Struct. E. London

In ordinary reinforced concrete permanent deformation takes place; thus cracks remain visible on removal of the load. A certain breathing of cracks was shown already by Professor E. Probst in a film at the Congress in Vienna in 1928. Also some healing of cracks has been ascertained. However, in spite of breathing and healing, too wide cracks exceeding a definite width (say 0.01 in i.e. 0.25 mm) represent a danger from the point of view of corrosion. Moreover in the event of excessive loading, permanent deformation of such magnitude occurs that the structure cannot be used further.

Quite different is the case with prestressed concrete. The outstanding characteristic of a prestressed concrete beam is its extraordinary resilience. Its behaviour is illustrated by typical load deflection lines obtained from beam tests made by the London & North Eastern Railway, London (L. N. E. R.) on prestressed concrete sleepers some of which had been in the track for 2 1/2 years (1) (fig. 1).

A sleeper is strained by tension on its under side below the chairs near its ends, and on the upper side in the centre portion; the magnitude of the stresses depending on the packing of the ballast. In view of this strain, the imparted pre-compression stresses are almost uniform over the whole section as indicated in fig. 1. The effective concrete stresses were approximately 1 650 to 1 800 lb per sq. in (115 to 125 kg/cm²). The numerical

(1) See *Concrete and Constructional Engineering*, April and May 1947, London.

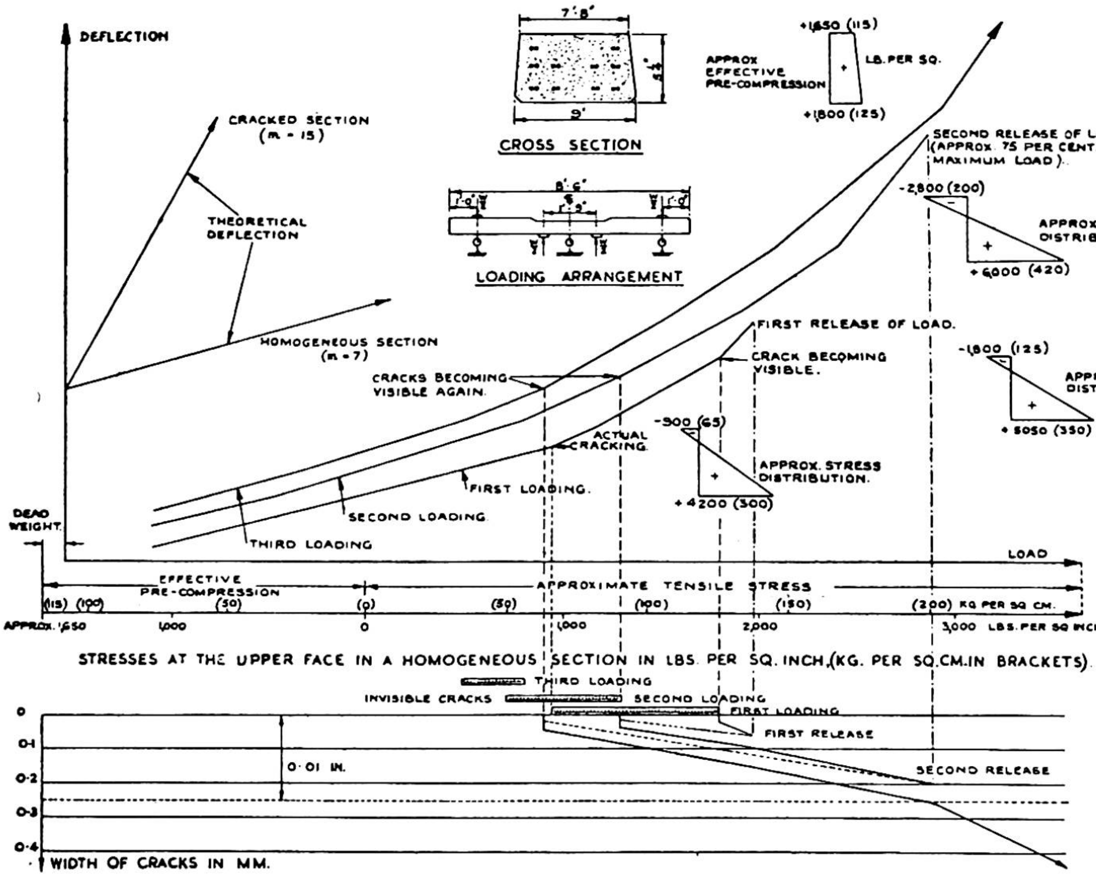

Fig. 1. Typical load deflection curves L. N. E. R. tests.

values of load and deflection are not plotted, but the concrete tensile stresses in a homogeneous section at the upper fibre are indicated.

The sleepers were loaded as simply supported beams, two point loads acting upwards, as indicated in fig. 1. The cycle of loading was to increase the load to somewhat above half of the ultimate load, then to reduce it nearly to zero, increase it again to 70 to 90 per cent of the ultimate load, to reduce it again nearly to zero and finally to increase it until the sleeper failed.

It can be seen from fig. 1 that before cracking occurs, the load-deflection line is approximately parallel to the theoretical line for a homogeneous section computed for a modular ratio $m = 7$. It can be assumed that very fine hair cracks developed at a load slightly in excess of that load for which a change in the inclination of the deflection curves is noticeable. These cracks, invisible to the naked eye, developed at a loading corresponding to a concrete tensile stress of approx. 900 lb per sq. in (63 kg/cm^2) but they became visible only at a loading corresponding to a much higher tensile stress, e.g. 1 800 lb per sq. in (126 kg/cm^2).

After cracking, the deflection curve is almost parallel to the theoretical line, calculated according to the standard method for a modular ratio $m = 15$ and neglected concrete tensile zone.

A cycle of loading, carried out as indicated in fig. 1 shows that after reduction of the load, the inclination of the deflection line is again parallel to that of the stage before cracking. This occurs even after the second

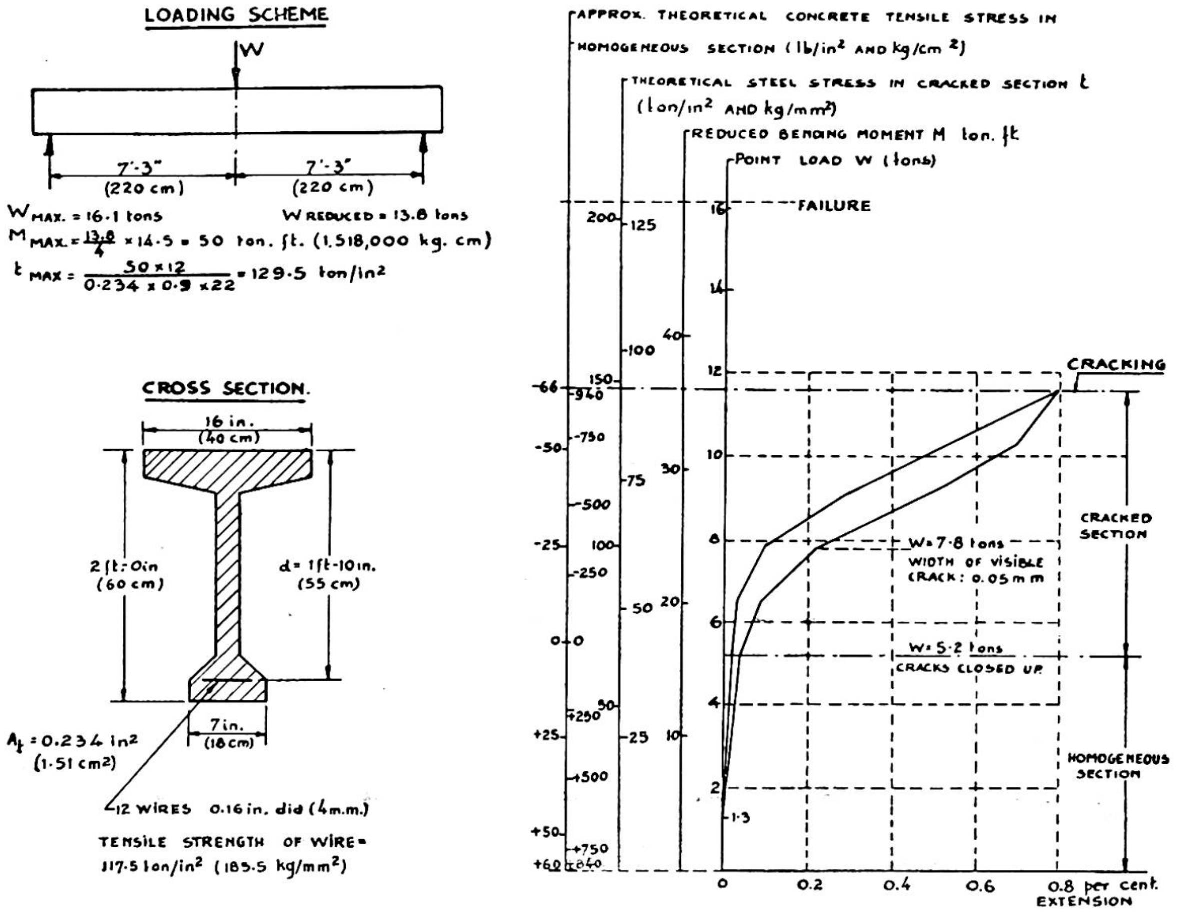

Fig. 2. Fine measurements of extensions over cracks according to E. M. P. A.
Note : These extensions have been measured after cracks had occurred.

reduction of load at 70 to 90 per cent of the ultimate load, which corresponds to a nominal concrete tensile stress in a homogeneous section of 2 500 lb per sq. in (175 kg/cm²) or more.

In fig. 1 is also shown the size of the widest cracks, observed during the tests. A width of 0.01 in (0.25 mm) is plotted for comparison, as this is generally accepted as the width of cracks which may develop in ordinary reinforced concrete and is therefore considered as permissible. It is seen that such a width occurs in fig. 1 only under loads exceeding 70 per cent of the ultimate load.

From fig. 1 the remarkable properties of prestressed concrete are clearly seen, notably the fact that cracks become invisible on reduction of the load, an entire closing of the craks apparently being ensured, when the concrete tensile stresses are reversed into compressive stresses. At this stage, the pre-compression which was previously temporarily interrupted at the cracks, is again in force along the entire length.

The cracks close, in fact, completely, as can be seen from fig. 2, representing the results of fine measurements of extensions of the steel carried out by the E. M. P. A., Zurich, for the Swiss Federal Railways. In this figure the cross section of the prestressed beam and the loading arrangement are indicated. The point load at failure of 16.1 tons is, according to E. M. P. A., reduced to 13.8 tons to obtain the actual bending moment at failure which would occur if two spaced point loads were acting instead of one, ensuring a bending moment of definite magnitude at the centre. The theoretical steel stresses in a cracked section, indicated in fig. 2 are based on this reduced load, whereas the approximate concrete tensile stresses in a homogeneous section are based on the bending moment corresponding to cracking.

The graph which is a mean of several measurements represents a hysteresis curve, no permanent extensions remaining at removal of the load. The cracks thus completely close at a concrete stress approx. zero. On the other hand the tests of the L. N. E. R. London mentioned before as well as tests carried out by the E. M. P. A., Zurich, have shown that an entire closing of cracks cannot be ensured when repetition of the maximum load, causing fatigue, takes place.

A special feature of prestressed concrete beams, reinforced with high strength wire, is the fact that in many cases the wire breaks at failure, the maximum theoretical steel stress being nearly always in excess of the tensile strength or at least equal to it, even if the wire does not break. For example, the stress at failure according to fig. 2 is 10 per cent in excess of the tensile strength of the wire. This excess of strength is not limited to prestressed concrete but occurs also with ordinary reinforced concrete, if a high degree of adhesion is ensured and the bond is destroyed in the immediate vicinity of the cracks only.

The author has tried to explain this phenomenon by a stress redistribution in the crack; in this connection it must be acknowledged that Dr. h. c. L. Herzka of Vienna suggested in 1935 that the behaviour in a crack of a reinforced concrete member may be similar to the stress concentration in a notch of a steel bar. The author has investigated this idea and shown a comparison with Professor Timoshenko's studies on stress concentrations in holes, published on the occasion of the Congress of Applied Mechanics in Zurich 1932. Another diagram was shown at the 2nd Congress of the Association in Berlin [2]. From this figure it is seen that the strength increases when the length of a notch is reduced. This phenomenon of apparently increased strength can be explained by a greatly reduced contraction in a short notch so that, in fact, the ultimate stress related to the net area is in all cases the same.

In a reinforced concrete section in which an efficient adhesion is ensured between concrete and steel near to a crack, a similar behaviour may be assumed to that in a notched bar.

When drawing the conclusion from the behaviour of prestressed concrete, discussed in connection with figs. 1 and 2 the following may be said :

(1) In constructions in which repetition of the maximum load occurs including impact (such as railway bridges, sleepers, and certain factory floors), the development of any cracks ought to be avoided and only compressive stresses should be allowed at present under working load.

(2) In constructions in which the maximum load occurs only occasionally (such as road bridges, poles, roofs, or certain floors), there is no need to exclude the appearance of concrete tensile stresses, even occasionally exceeding the modulus of rupture, provided that, under dead weight, concrete tensile stresses do not occur.

This limitation has the effect that any fine hair cracks which may

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(2) See Figure 15, *Yield Limits and characteristic Deflection Lines* by Prof. RINAGL; *Preliminary Publication*, 2nd Congress Intern. Assoc. for Bridge and Struct. Eng.

have developed instantaneously under an occasional maximum load, will close immediately when the load is removed. Nominal tensile stresses of 600 to 1 200 lb per sq. in (42 to 84 kg/cm^2) and even more may be considered as permissible in such a design according to the report of the L. N. E. R. of 1947.

A member in which concrete tensile stresses appear under working load may be called partially prestressed as distinct from a fully prestressed member in which only compressive stresses occur under working load. No cracks will occur under sustained loading, if the tensile stress is below 75 per cent of the modulus of rupture.

Acknowledgment. The Author would like to express his thanks to the Civil Engineer of the Eastern Region of British Railways London (for leave to use the results of extensive investigations upon prestressed sleepers tested as simply supported beams), and to the Chief Engineer of the Swiss Federal Railways, Berne, as well as to the E. M. P. A., Zurich, (for permission to use the fine measurements described in fig. 2).

Résumé

Se basant sur les résultats d'essais anglais et suisses, l'auteur discute les propriétés principales du béton précontraint. Celui-ci se comporte comme matériau homogène *avant* sa fissuration et comme le béton armé ordinaire *après* sa fissuration. Après décharge, les fissures se referment entièrement dès changement de sens des tensions et la construction est de nouveau homogène. Cette propriété élastique remarquable constitue une des différences capitales par rapport au béton armé.

Cette propriété permet d'admettre, comme pour un matériau homogène, jusqu'à fissuration, une distribution linéaire, même s'il se présente auparavant quelques fines fissures, à condition toutefois de n'avoir aucune sollicitation de fatigue ou de choc. Dans le cas contraire, comme par exemple pour les ponts-rails ou traverses de chemin de fer, il faut éviter les fines fissures et même toute sollicitation de traction dans le béton.

Zusammenfassung

Gestützt auf englische und schweizerische Versuchsergebnisse, werden die wichtigsten Eigenschaften von vorgespanntem Beton besprochen : Er verhält sich wie ein homogenes Material *vor* der Rissebildung und wie gewöhnlicher armierter Beton *nach* der Rissebildung. Bei der Entlastung schliessen sich die Risse vollständig, sobald die Zugspannungen verschwunden sind und der Bauteil verhält sich wieder wie einer aus homogenem Material. Diese bemerkenswerte Elastizität ist einer der Hauptunterschiede gegenüber gewöhnlichem Eisenbeton.

Es kann daher wie in einem homogenen Material bis zur Rissebildung eine geradlinige Spannungsverteilung angenommen werden, auch wenn sich schon vorher einige temporäre, feine Risse gebildet haben. Dies aber nur unter der Voraussetzung, dass keine Ermüdung und keine Stossbelastung vorkommt. Wenn dies nicht der Fall ist, wie z.B. bei Eisenbahnbrücken und -Schwellen, sollte das Auftreten von feinen Rissen, besser überhaupt das Auftreten von Zugspannungen im Beton vermieden werden.

Summary

On the basis of British and Swiss test results, the essential feature of prestressed concrete is discussed : it behaves like a homogeneous material before cracking and like ordinary reinforced concrete after cracking, but when the loading is reduced, the cracks close entirely on reversal of the tensile stresses into compressive stresses, whereupon it behaves again similarly to a homogeneous material. This remarkable resilience is one of the main distinctions from ordinary reinforced concrete.

A straight line stress distribution in homogeneous material can therefore be assumed below cracking stresses, independently of whether repeated temporary fine cracks have already developed or not, provided fatigue and impact strain (as with railway bridges and sleepers) do not occur. In the latter case the development of fine cracks and even the occurrence of any concrete tensile stresses ought to be avoided.

IIb4

Considérations économiques sur le béton précontraint

Die Wirtschaftlichkeit von vorgespanntem Beton

The economy of prestressed concrete

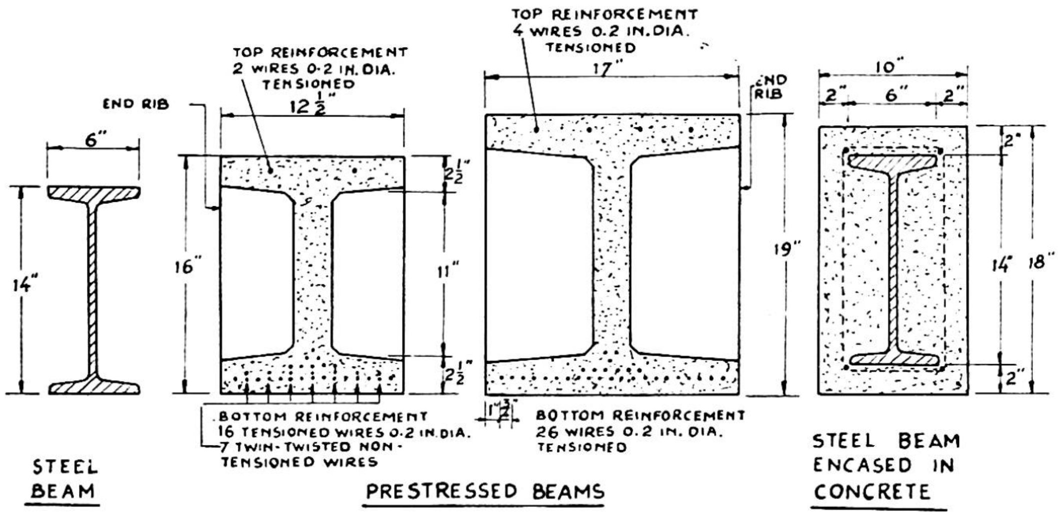
PAUL WILLIAM ABELES
D. Sc. (Vienna), M. I. Struct. E. London

The cost of prestressed concrete work is made up very differently in the cases of pre-tensioning (when the prestress is transferred to the concrete by bonded wires which were tensioned previous to casting) and post-tensioning (i.e. when the prestress is obtained by reaction against the hardened concrete, the steel being non-bonded at tensioning).

Pre-tensioning relates particularly to precast reinforced concrete; consequently, the main consideration for pricing is that of precast concrete work. The cost per unit volume of precast concrete depends particularly on the amount of steel required (e.g. 6 lb per cu. ft corresponding to 100 kg/m^3) and on the amount of depreciation of the mould taken into account. Thus, the price of precast reinforced concrete placed in position varies, within a wide range, for various products, if in addition to differences in steel quantities, shape and depreciation of mould, also different distances for carriage are taken into account.

On the whole, the steel consumption of prestressed concrete per unit of concrete is much less than that of ordinary precast reinforced concrete, even if in view of the higher permissible stresses, the concrete section is reduced. On the other hand, the price of high strength wire is considerably higher than that of mild steel, and the cost of tensioning must also be taken into account. Considering these points, it can be said that the unit price of prestressed concrete should not greatly exceed that of ordinary reinforced concrete (say up to 20 per cent). This depends mainly on an economical use of moulds and on a suitable arrangement for tensioning. It must be borne in mind that the price depends greatly upon the output, which, in turn, is dependent on the demand.

In fig. 1, a steel joist $14'' \times 16'' \times 46$ lbs is compared with two prestressed concrete members, one designed as a partially and the other as a fully prestressed beam. The stresses and material consumptions are seen



DEAD WEIGHT	46	90	120	210	LB. PER FOOT
STEEL WEIGHT	46	3.2	3.0	APPROX. 50	
$P_t + P_t'$	—	APPROX. 80,000	APPROX. 120,000	—	LB.
STRESS-DIAGRAMS	18,000 + − -18,000	-170 + +1940 AT TRANSFER; 1500 + 0 AT 0.57W; 2,725 + − -1220 AT W	0 + 2000 AT TRANSFER; 1,700 + 0 AT WORKING LOAD W	STEEL 18.000 + − -18,000; CONCRETE 1,550 + − -1,550	LB. PER SQ. IN.

Fig. 1. Comparison of stresses and material consumptions.

in fig. 1. The fourth member is a steel joist embedded in concrete wherein the additional carrying capacity of the concrete is neglected as is usual in railway design.

If a price of £ 40 per ton steel and 18/— per cu. ft precast concrete is taken into account, including transport and placing, then the following comparative costs per ft run are obtained; Steel Beam — 16/6 d. Partially Prestressed Beam 11/7 d. and Fully Prestressed Beam 15/3 d., the fourth member being obviously much dearer than the steel joist itself. It is seen that a fully prestressed member is cheaper than a steel joist, but the construction depth is increased. On the other hand, with a partially prestressed member both the depth and the costs are further reduced. However, this construction is unsuitable for certain purposes where permanent freedom from fine cracks must be ensured, but could readily be employed, for, example for roofs, floor, or transmission poles in which the maximum load occurs only occasionally, any fine hair cracks closing up in this special instance if the load is reduced to 57 per cent of the entire working load, when only compressive stresses occur in the section [1].

In the case of post-tensioning, the cost can be more clearly assessed than with pre-tensioning, since the main items for stretching the wires are represented by the end anchorages, the preparation of the cable and the hole or groove (afterwards to be filled with cement mortar) and the tensioning operation itself, the cost varying only slightly with the

[1] See the author's contribution : *The behaviour of prestressed concrete at cracking. Conclusions for Design.*

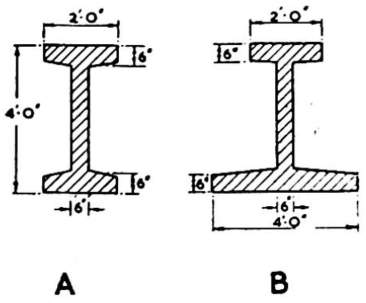


Fig. 2. Girders of types A (partially prestressed) and B (fully prestressed).

Girder "A"	£ s. d.	Girder "B"	£ s. d.
Concrete 320 cu. ft **a** 5 s.	80 0 0	430 cu. ft **a** 5 s. . .	107 10 0
Wire 0.55 tons **a** £ 80.—	44 0 0	0.6 tons **a** £ 80.— .	48 0 0
Tensioning, etc. 88 wires **a** 11 s. 3 d. (including anchorages)	49 10 0	138 wires **a** 11 s. 3 d.	77 12 6
Mould, used for 10 girders, say . . .	20 0 0	Mould, say . . .	23 0 0
Hoisting of girder 20 ft, say	30 0 0	Hoisting, say . . .	35 0 0
Total £	223 10 0	Total £	291 2 6

TABLE I. *Cost per girder.*

length of construction. Generally, the following costs apply for a group of wires, be it a round cable of the Freyssinet pattern or the Magnel-Blaton sandwich cable.

$$\text{Cost of prestressing} = C + H + A + J + T = a + b\,L.$$

C is the cost of the cable excluding the tensioned wires, but including spacers and placing, H the cost for preparing the hole or groove, and later filling it with cement mortar, A represents the cost for anchorage for the groups of wires at both ends, J is the cost for supplying the jack for the duration of tensioning the group of wires and T is the labour cost for tensioning. The entire costs are to a certain extent dependent on the length L of the member, a and b being constant values, in which a is normally of much greater importance than b L. Obviously, these values will differ for different type of cables and anchorages and may vary for different jobs, dependent on the influence of transport cost, travelling expenses, and the extent of the work. On the whole, post-tensioning will be economical for relatively long members which are produced near the site and hoisted into position.

Fig. 2 is a comparative example of girders of 80 ft span capable of carrying a load of 1 200 lb per ft, 40 per cent of which is live load, the depth being limited to 4 ft. A steel truss complying with these conditions would require a cross-sectional area at the top and bottom of approx. 16 sq ft resulting in a steel requirement of approx. 6 tons; this would cost say £ 360 per girder placed in position based on a steel price of £ 60 per ton, it being assumed that no lateral bracing is required (which would mean an increase in cost).

Table I gives details of cost of a partially prestressed girder A in which only compressive stresses occur under dead weight and of a fully prestressed girder B on the assumption that at least 10 girders are built. The cost of prestressing is taken rather high with 11/3 d. per tensioned wire of 0,2 in dia.

From Table I it is seen that a prestressed concrete girder is much cheaper than a steel truss of the same limited depth. Even in the event of some additional mild steel reinforcement being provided, the cost would be increased only by say 10-15 per cent, if approximately the same quantity of mild steel as wire were used.

The position becomes, however entirely different if the depth of the girder is not limited. Assuming for example the depth may be doubled, then the cost of the steel truss would be almost halved, to say £ 200 per girder. However, it is rather doubtful whether the costs of the prestressed beams could be appreciably reduced by increasing the depth. While the cost of the wire and that for tensioning, including anchorage, would be reduced, that for the concrete, mould and hoisting would be increased. Thus, a reduction of more than, say, 15 per cent, could hardly be expected for a girder of unlimited depth, resulting in £ 190 and £ 257 respectively for designs A and B.

It should be noted that in this special case, design A is still competitive and could be employed without hesitation in countries in which the full live load (snow) only seldom occurs.

From the foregoing it is seen that greater economy will be obtained by using prestressed concrete in cases of limited depth. This is of particular importance for floor constructions and road bridges over railways (in order to reduce the approach roads to a minimum). Fig. 3 shows two designs of road bridges which are being built in Great Britain to replace stone arches in order to provide clearance for an overhead collector wire in connection with the electrification of a railway line. A slab construction has been found most suitable to reduce the depth to a minimum and a combined construction has been chosen, as shown in design 1 in which the precast prestressed component with bonded wires represents the permanent shuttering of the slab. It is designed to carry the dead weight of the entire slab without any support and, in conjunction with the in-situ concrete of the slab, the dead weight of the deck construction and the live load. Under dead weight no tensile stresses occur in the prestressed members even after the greatest possible losses of the initial prestress have taken place; whereas under maximum live load (which however occurs in a road bridge very seldom) tensile stresses are permitted in accordance with the recommendation given in (1).

In this case the prestressed component is approx. 1/3 of the entire concrete consumption. An alternative design for encased steel joists was prepared in which the joists had to be reduced to 14 in (i.e. the same depth as the prestressed beams) in order to allow sufficient concrete cover. However, the additional carrying capacity of the concrete encasing has not been taken into account as is the practice in railway design. According to the lowest tender, the encased steel construction of one bridge would have been approx. 50 per cent dearer than the design 1 (fig. 3) taking into account only the concrete slab; and approx. 30 per cent, taking into account the whole bridge superstructure including parapet and deck.

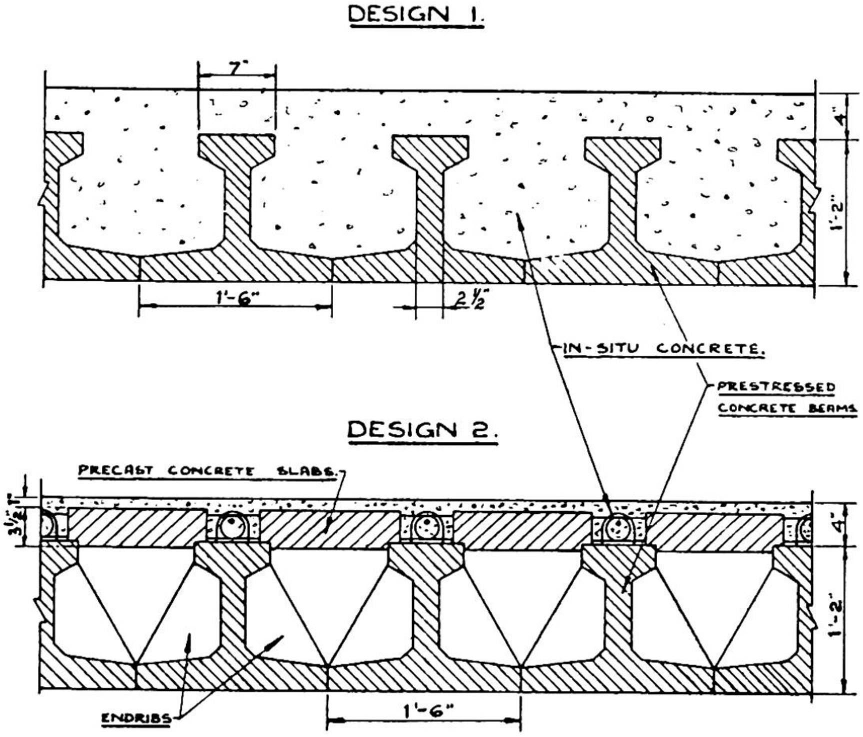


Fig. 3. Cross sections of two bridge designs.

Instead of 14 1/2 tons mild steel (13 3/4 tons joists and 3/4 ton steel bars) only 3 tons (1/2 ton hard drawn wire plus 2 1/2 tons mild steel bars) are required for the superstructure of the bridge in question. Thus, both reduction of cost and considerable saving of steel have been attained.

Design 2 of fig. 3 shows a cross-section of another solution, using the same type of prestressed beam in combination with precast slabs. Co-operation is ensured by a strip of in-situ concrete provided over the top of the joists into which links protrude from the individual precast elements, in addition to a continuous in-situ topping. This design is employed for bridges for which a lesser carrying capacity is required and a reduction of the in-situ concrete to a minimum is desirable in view of their isolated location. The saving in weight and cost is even greater in this case if compared with the conventional solution of a solid slab containing encased steel joists. (Acknowledgment : Fig. 3 is published with permission of the Civil Engineer, of the Eastern Region, British Railways, Kings Cross, London.)

In connection with the question of economy, fig. 4 may serve to discuss the development of a prestressed concrete floor construction. The Norwegian J. G. F. Lund's suggestion of 1907 was not successful similar to all other proposals of prestressing of that time, because the initial tensioning stress was too low and soon became ineffective. However, even if an effective prestress had been obtainable, this solution would not have been very suitable or economical.

Though the prestress is transmitted at post-tensioning by washers (as seen in the fig.) it would not be possible to prefabricate individual members but necessary to place the blocks on a centering, since the bars are located in holes formed by interlocking grooves of adjacent parts. Morever, it would be impossible to obtain an additional bond, since the mortar filler

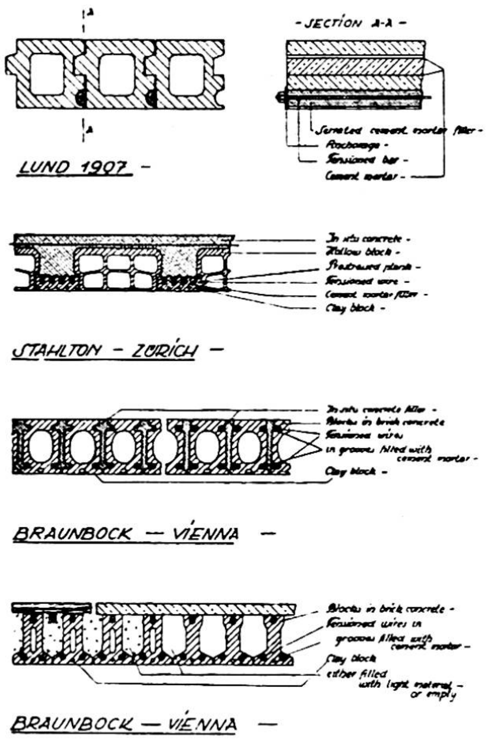


Fig. 4. Development of a prestressed concrete floor construction.

would have to be inserted into the hollows, before tensioning and the bond between mortar and bar would be broken at tensioning.

Compared with the proposal by Lund, the "Stahlton" floor, using prestressed planks, represents a great technical and economical improvement, the prestressed precast component being reduced to a small proportion. These planks can be produced in a small factory in which wires are tensioned over a long length and severed at the ends of the individual planks, after a mortar filler inserted in the individual grooves of clay blocks containing the wires, has hardened. The " Stahlton " floor has proved to be competitive with other non-prestressed floors apart from saving steel.

Though the manufacture of the " Stahlton " plank is highly economical and suitable for cheap mass production in a small factory, a certain plant is required. Mr. E.Braunbock of Vienna has developed a method for which no plant is required. Blocks of clay or brick concrete are prestressed at the site by means of a simple screw jack. Either they are only temporarily post-tensioned and the endplates are removed after the cement mortar filler (which completely surrounds the tensioned wires placed in grooves) has hardened; or relatively cheap anchor means are provided which remain permanently. The grooves may be filled before placing the individual member or afterwards, when the interspaces between adjacent members, together with the grooves are filled with cement mortar. These solutions allow the prestressing to be carried out at the site without requiring a factory. Thus, the extent of transport and handling is reduced, but the precast component is rather large which may offset the saving due to this reduction. It will therefore depend on certain circumstances whether this solution is really cheaper than the " Stahlton " floor. However, both these proposals illustrate the development of an economical floor construction.

When summing up, it can be stated that prestressed concrete is especially competitive when of limited depth. Precast members should be rather light in view of cost of transport and handling; thus, combined sections are always more economical. Of particular importance is an economical use of moulds. Post-tensioning allows new applications of concrete in cases in which ordinary reinforced concrete is unsuitable.

Résumé

L'auteur distingue deux cas : *Pré-tension* (lorsque les fils sont tendus avant le bétonnage) et *Post-tension* (lorsque les fils sont tendus après durcissement du béton). La pré-tension est en général limitée sur des éléments de construction alors que la post-tension est applicable sur des grandes constructions, à exécuter sur le lieu d'érection où a lieu leur mise en place par des engins de grande puissance. L'auteur donne des exemples comparatifs pour les deux modes de construction et montre les avantages économiques d'éléments partiellement précontraints.

L'auteur décrit ensuite deux types de passages supérieurs de ponts-routes construits en Angleterre. Pour terminer il donne le développement des planchers en béton précontraint.

Zusammenfassung

Zwei Fälle werden unterschieden; *Vorheriges Spannen* (wenn die Drähte gespannt werden, bevor der Beton gegossen wird) und *Nachträgliches Spannen* (wenn die Drähte gegen den erhärteten Beton gespannt werden). Vorheriges Spannen ist im wesentlichen beschränkt auf Eisenbeton-Fertigteile, während ein nachträgliches Spannen auf grosse Konstruktionen angewendet werden kann, die am Ort oder ganz in der Nähe der Baustelle ausgeführt und mit Hebezeugen in die endgültige Lage gebracht werden. Der Verfasser bringt vergleichende Beispiele für beide Arten und weist auf die grossen wirtschaftlichen Vorteile von teilweise vorgespannten Gliedern hin.

Dann bespricht der Verfasser zwei Ueberbautypen von Strassenbrücken, die in Grossbritannien über einige Eisenbahnlinien gebaut werden. Schliesslich wird noch die Entwicklung der Decken aus vorgespanntem Beton beschrieben.

Summary

Two cases are distinguished : pre-tensioning (when the wires are tensioned before the concrete is cast) and post-tensioning (when the wires are tensioned against the hardened concrete). Pre-tensioning is mainly limited to precast concrete, whereas post-tensioning is suitable for large constructions manufactured in place or near the site and boisted into position. Comparative examples for both cases are presented and the great economy of partially prestressed members is seen.

Two general designs of superstructures of road bridges are shown, which are being built over certain railways in Great Britain; also, the development of prestressed floor construction is illustrated.

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IIb5

Essais sur des cadres en béton précontraint réalisés pour des bâtiments à étages

Versuche an vorgespannten Betonrahmen von mehrstöckigen Gebäuden

Tests on precast prestressed concrete frames in multi-storey buildings

by the late K. W. MAUTNER
D. Sc., M. I. Struct. E., M. S. C. E. (France)

The substitution of precast reinforced concrete elements for structural steel work for multi-storey buildings meet with practical and economic difficulties.

The difficulty lies in connecting the elements on the site by means of protruding reinforcement encased in situ, this connection requiring much labour and time.

Attempts have been made to mould complete reinforced concrete frames with two or three legs and to transport them to the site, lift them in position and connect them with the frames below by means of pinjoints or the like.

Transport difficulties, however, are considerable here.

The underlying idea of the prestressed precast concrete frames is as follows :

Applying the Freyssinet method of non-bonded but anchored cables, the latter to be grouted in after completion, frames are constructed from prismatic beams and by elements. These elements have no protruding parts except short steel plates for connecting the frame of one storey to the storey above by simple welding, which plates are connected to the elements by short welded mild steel bars. The precast elements are of simple shape and contain only enough M. S. for handling purposes.

On the site the frames are assembled by tensioning and anchoring the cables, producing a pre-compression sufficient to ensure at each joint a tensile stress nearly nil.

Should a load higher than the design load occur the butt joint would open and close again after disappearance of the load.

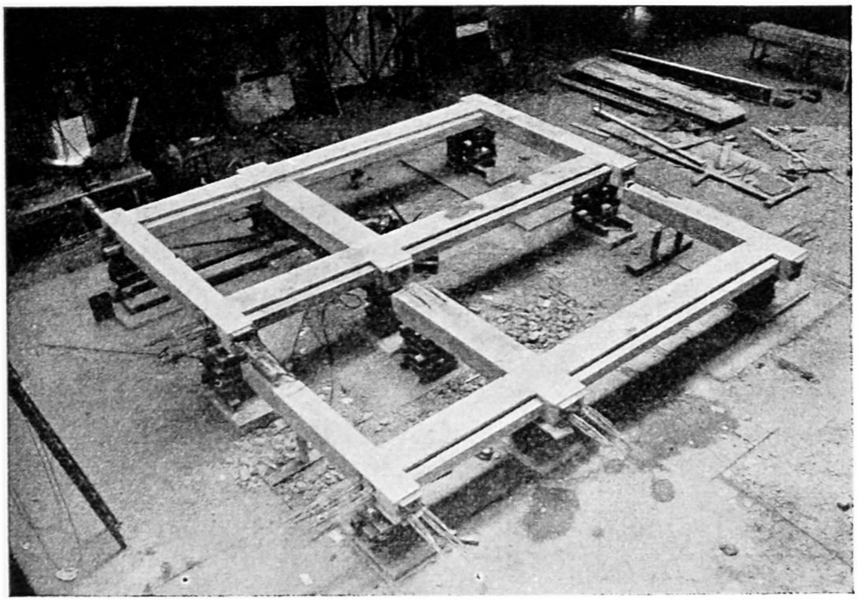


Fig. 1. Two separated frames, prestressed but not yet assembled.

The system was planned by the author for a London County Council 10 storey block of flats. Frames were at 14 feet crs., the slabs, spanning 14 feet and 7 feet respectively.

Extensive full scale tests were carried out at the initiative of Messrs. Structural & Mechanical Development Engineers, Ltd., Slough, Bucks., by the Buiding Research Institute under the guidance of Dr F. G. Thomas.

The strain measurements were made by a number of telescopic mirror gauges and checked by means of the acoustic strain gauge developed by the Building Research Institute.

The frames were loaded by hydraulic jacks placed in the centre of a beam, spans for the live load and a horizontally acting jack representing wind load on the two uppermost storey of a 10 storey frame.

As it was only possible to carry out the test in the flat position on a two storey frame, the action of the dead weight of the columns was

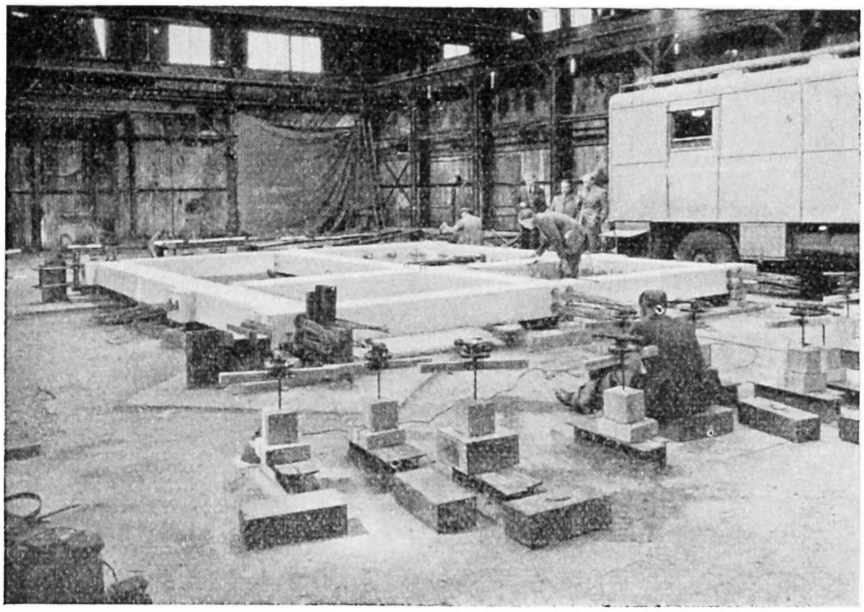


Fig. 2. Two frames assembled and under loading test with mirror deflectometers and accoustic strain gauges.

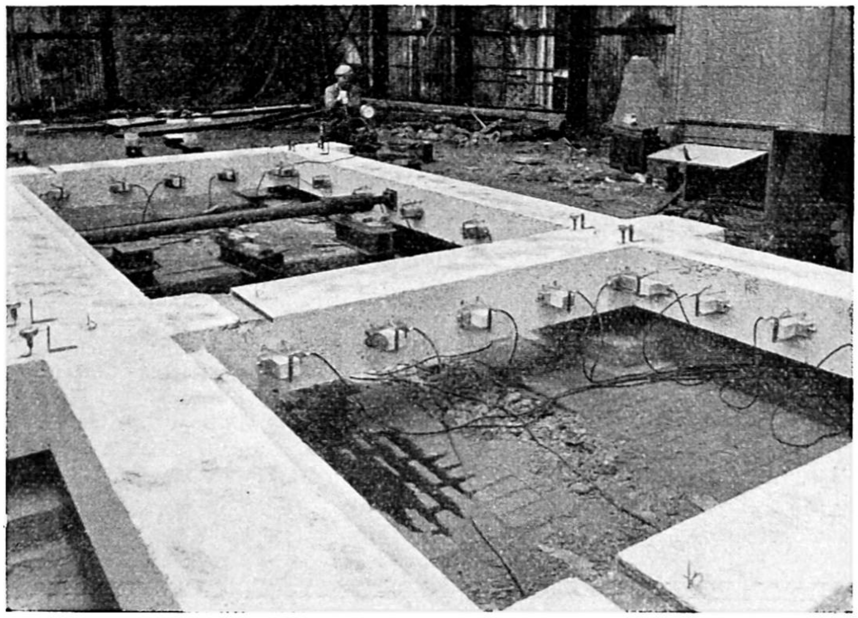


Fig. 3. Detail of the measuring apparatuses with accoustic strain gauges.

neglected and therefore the results were on the unfavourable side with regard to the tensile stress in the columns.

The tests for wind load were carried out with the design wind load of 25 lb/sq. ft on one side only, by inserting a jack on the uppermost corner, the force being of 6.5 tons, increased gradually up to 13.6 tons.

The vertical design load for the beam test was estimated to correspond to a single load of 5.6 tons applied at mid span of the longer beam. This was gradually increased up to 10.9 tons.

The results of these tests can be summarized as follows : No cracks occurred in any of the members at the design load but the joint between the centre columns and the top beam opened at one end by 0.004″.

At a load of 9.2 tons the opening of the joint between the centre column and the top beam became 0.01″ and slight cracks occurred at each of the welded joints with maximum width from 0.004″ to 0.011″.

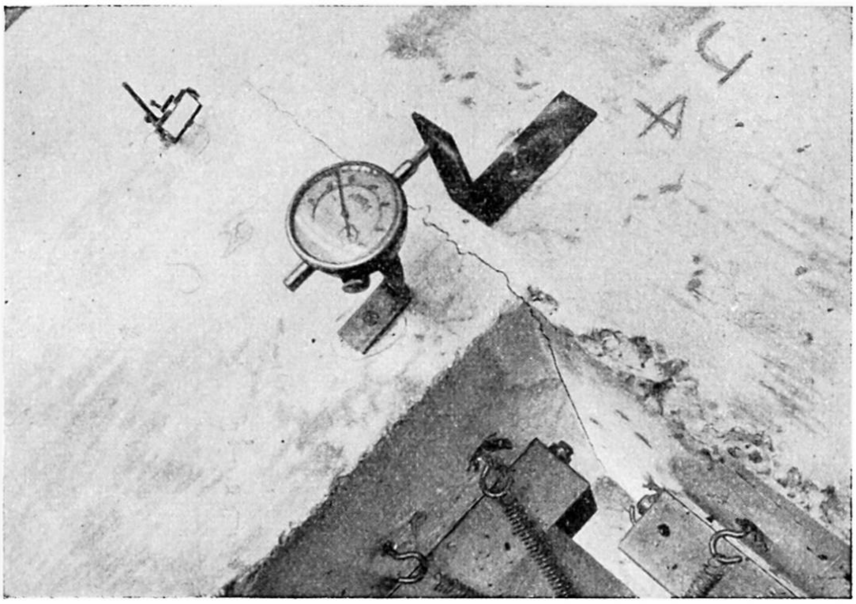


Fig. 4. Opening of one butt joint under excess load.

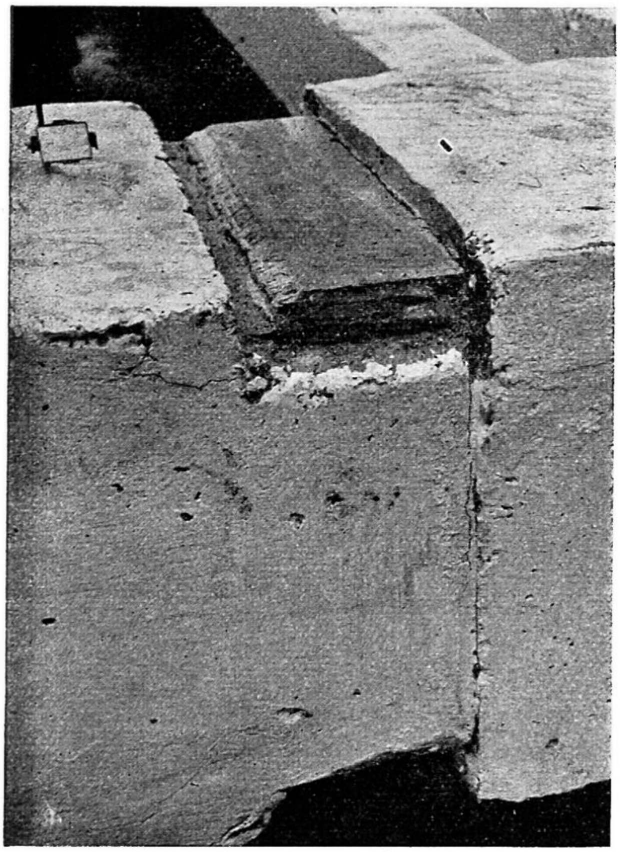


Fig. 5. Crack under excess load at welded joint.

After the beam test, when lateral loading of the frame was continued a crack in the upper column to which the load was applied was observed at about 1 1/2 times the design load. With increasing loading openings appeared at all beam column junctions, and at twice the design load cracks or opening of the joints varying between 0.001″ and 0.008‴ appeared in the column on which the wind load of 13.6 ton i. e., double the design wind load was applied. *On removal of the load all cracks in the concrete disappeared.* The conclusions drawn by the Building Research Institute were :

1. The strength and stiffness of the frame at the time of the test were adequate for the load for which it was designed;

2. Cracks are unlikely to occur in the member of the frame work unless the frame work is *seriously* overloaded. Cracks developed during a period of overload will disappear on its removal.

The test showed further that bending moments obtained by analysis agree with bending moments calculated from measured strains except that the welded steel joints showed greater strains than the adjacent prestressed concrete sections.

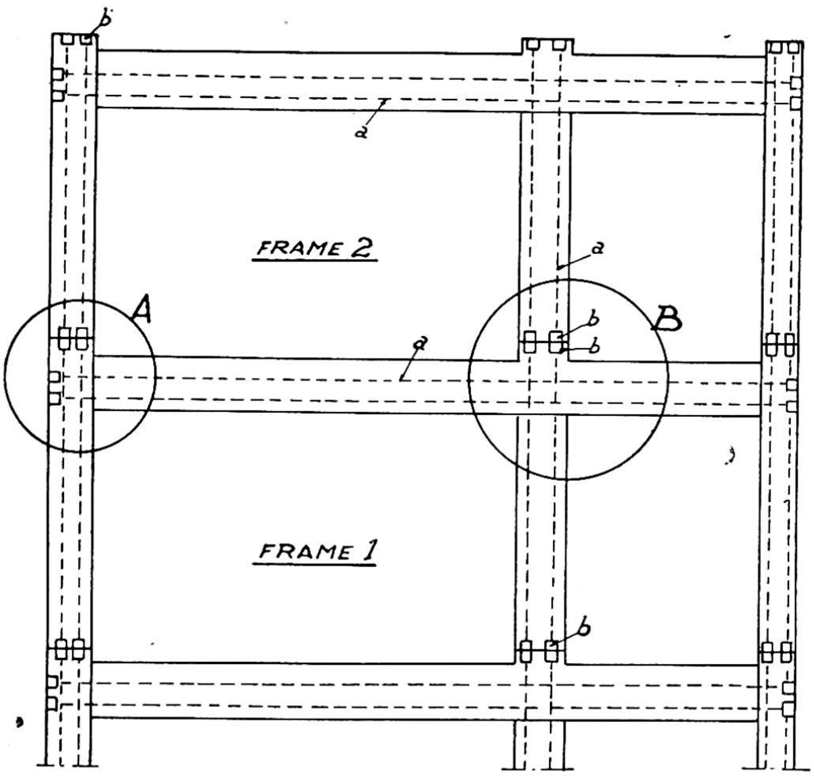


Fig. 6. Schematic drawing of two frames for testing :
a. – Non-bonded cables;
b. – Anchored cables.
For details A and B see figures 11 and 12.

This resulted in a shifting of the points of contraflexure towards the welded joint thereby slightly increasing the bending moment on the top.

When comparing this system with the usual system of steel framework assembled by cleats and bolts or rivets it should be borne in mind that the steel frame work usually is not designed with the assumption of completely stiff nods but as a semi-rigid system taking in account the yield of the connections. The assumptions therefore for which the prestressed concrete frames had been tested are more rigorous than that of the usual steel skeleton. Although the wind load applied was one sided and of great magnitude the openings and cracks generally closed after the removal of the load even when double the design load had been applied, except very fine cracks in the cover of the welded steel plates.

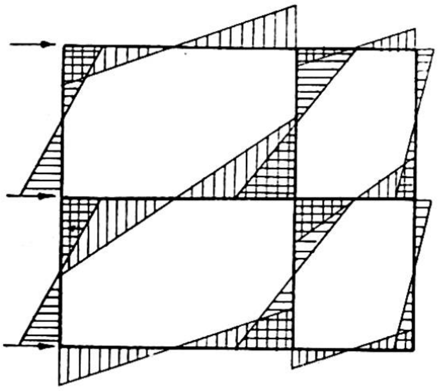


Fig. 7. Diagram of wind load moments.

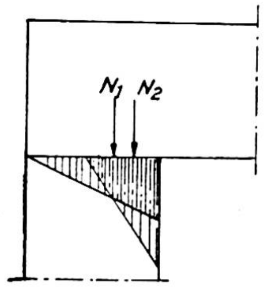


Fig. 9. Horizontal butt joint.

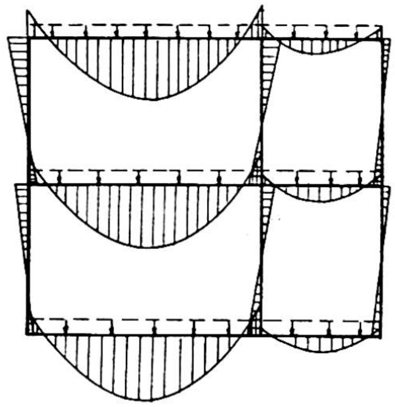


Fig. 8. Moments by S. I. loads.

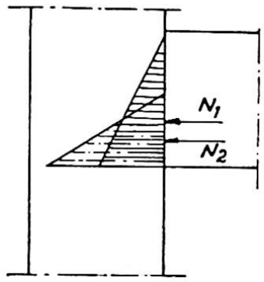
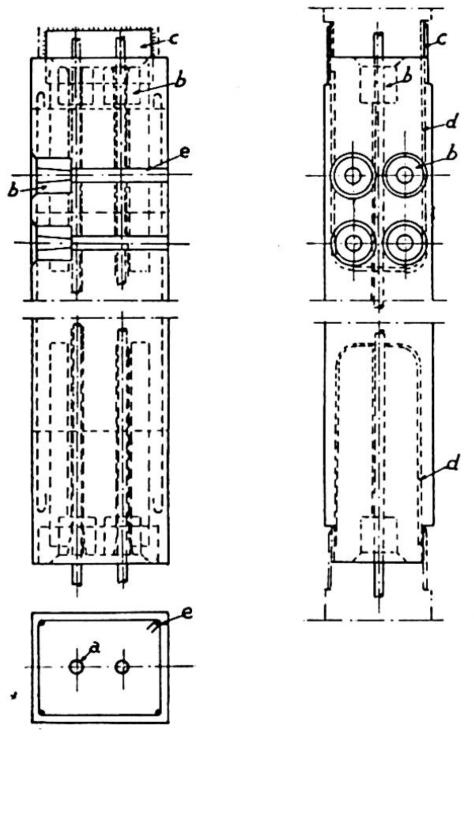


Fig. 10. Vertical butt joint.

N_1 : Normal force for designed load.

N_2 : Normal force for exaggerated load, with opening of the joint.



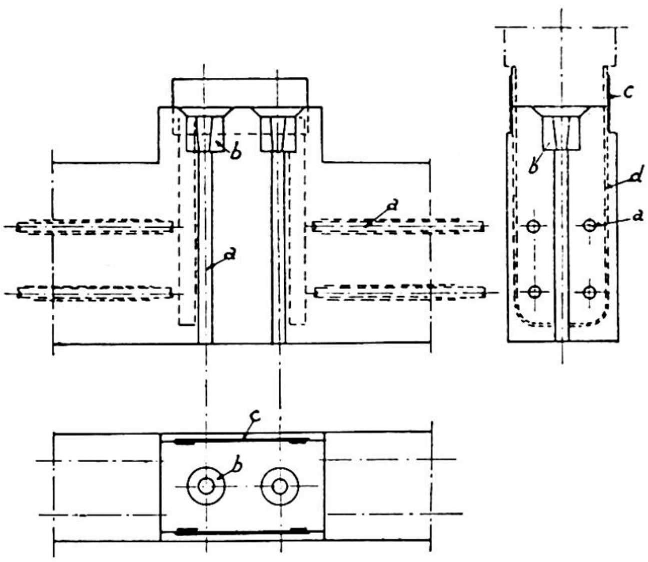


Fig. 11 (left). Detail « A » of figure 6 (plan and side elevation).

Fig. 12 (right). Detail « B » of figure 6 (plan and side elevation).

For practical applications it is preferable to apply the multiple frame system for the wind load only and to have the super imposed load supported by slabs spanning parallel to the wind frames. In this case the prestressing of beams and columns can be centrically applied as this is suitable for wind loads alternately on each side applied on one and the other side.

Résumé

En vue de la construction d'un bâtiment à dix étages en éléments préfabriqués en béton précontraint, des essais ont été poussés sur deux étages en grandeur nature.

Les éléments (poutres et colonnes) ont été bétonnés séparément puis assemblés en cadres à trois traverses d'après le procédé de Freyssinet, c'est-à-dire par des câbles à ancrage spécial.

Les essais ont montré la parfaite concordance avec les résultats des calculs pour les charges prévues; pour des surcharges importantes, il se produit des fissures se refermant après déchargement.

Le système utilisé constitue un développement du procédé bien connu de Freyssinet pour le montage de cadres à deux dimensions en éléments préfabriqués.

Zusammenfassung

Zur Abklärung der Aufstellung der Rahmen eines mehrstöckigen Gebäudes aus vorgespannten Fertigbetonelementen wurden Versuche an zwei Stockwerken des geplanten zehnstöckigen Gebäudes in natürlicher Grösse ausgeführt, und zwar für Wind- und Nutzlast.

Die Elemente, Balken und Säulen, welche einzeln betoniert wurden, wurden zusammengesetzt zu dreistieligen Rahmen nach dem Verfahren von Freyssinet, d. h. mittels unverwundener, speziell verankerter Kabel.

Die Versuche ergaben, dass das Verhalten der Rahmen mit der analytischen Berechnung übereinstimmt für die dem Entwurf zugrunde gelegte Last und dass bei einer grossen Ueberlastung Oeffnungen und Risse entstanden, welche sich aber wieder schlossen nach dem Entfernen der Last.

Das angewandte System ist eine Weiterentwicklung des bekannten Verfahrens von Freyssinet für die Montage von zweidimensionalen Tragwerken aus Fertigteilen.

Summary

For the purpose of erecting the frame work for multi-storey buildings by precast pre-stressed concrete elements, tests on a full scale have been carried out on 2 stories of a designed 10 storey building, for windloads and super imposed load.

The elements, beams and columns which had been precast, had been assembled to three leg frames by the Freyssinet method of non bonded especially anchored cables.

The tests have shown that the behaviour of the frames corresponded with the analytical computation for design load and that at serious overloading openings and cracks occurred, which however closed after removal of the load.

The system is an amplification of the known Freyssinet process of erecting structures by precast elements for two dimensions.