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Partially Prestressed Concrete Constructions, Built in the Eastern Region of British Railways 1948—1952

Teilweise vorgespannte Betonkonstruktionen, erbaut in der "Eastern Region" der Britischen Staatsbahnen 1948—1952

Les ouvrages en béton partiellement précontraint réalisés dans la partie est des chemins de fer britanniques de 1948 à 1952

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I. Theoretical Considerations

1. The essential features of partial prestressing

Prestressed concrete is a unique material and may exhibit any type of behaviour when designed accordingly, i.e.:

a) elastic with brittle failure;
b) elastic with slight plasticity before failure;
c) elastic with considerable plasticity before failure, and
d) plastic with slightly elastic behaviour at low stresses.

In Fig. 1, load deflection curves are shown for four types of structures considering under-reinforced sections with bonded wire for which the bending moment at failure depends solely upon the steel reinforcement [1, 2][1])

The ultimate force T_{ult} in the tensile reinforcement A_t at failure (bending moment M_{ult}) equals $\frac{M_{ult}}{a}$, where "a" is the lever arm of the inner forces and the following relation applies $T_{ult} = K_u \cdot A_t \cdot t_{ult}$, where t_{ult} is the tensile strength of the steel and K_u is a factor which will be unity if there is no distinct yield point and efficient bond is ensured.

The load deflection diagrams, Fig. 1, relate to beams of the same cross section having the same sectional steel area A_t and same working load W.

Beam a) is over-prestressed, i.e. cracking and failure occur simultaneously as with a brittle material; case d) relates to a non-prestressed beam while cases b) and c) are intermediate solutions.

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[1]) See Bibliography.

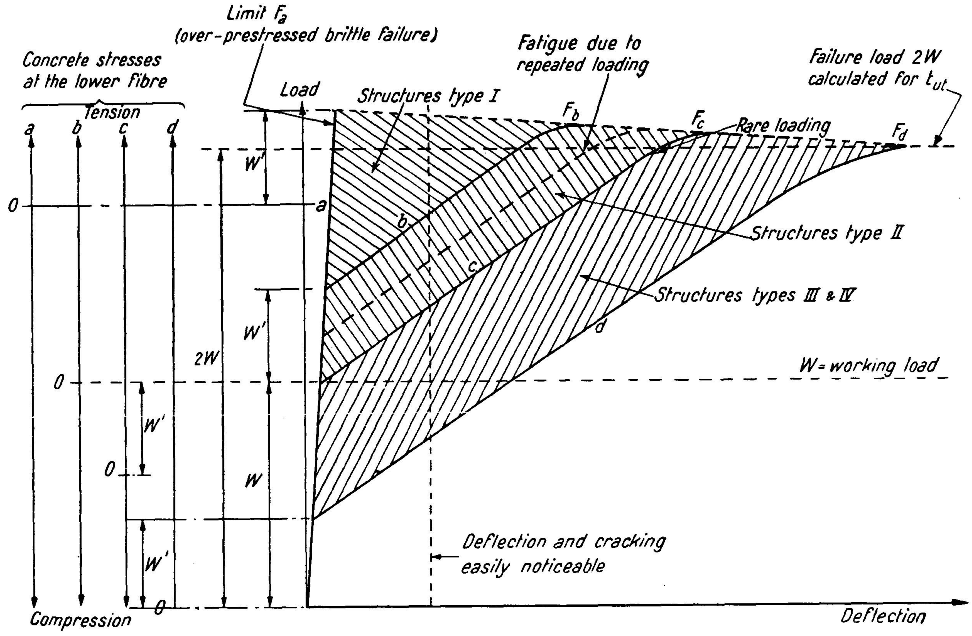


Fig. 1. Different types of structures, load deflection diagrams

In all instances the ultimate failure load F will be approximately the same, except that with higher percentages of reinforcement it may become larger if the prestressing force is increased, as indicated in Fig. 1. For low percentages the linear connection of the failure loads F_a to F_d may become horizontal. In the non-prestressed case d), cracks will develop under a load W' corresponding to the flexural strength of the concrete (modulus of rupture). In beam c), cracking is assumed to take place under the working load W; an effective pre-compression must therefore be ensured up to the loading $W - W'$.

Condition b) represents the lower limit of "full" prestressing, where permanent compressive stresses only are ensured under working load.

At the left hand side of Fig. 1 the resultant stresses at the tensile fibre due to prestress and working load are shown. In beam a) there are compressive stresses up to the failure load less W', while in beam b) the same conditions apply up to working load, in c) up to the working load less W' and in d) there are no induced compressive stresses at all at any time.

Type I are fully prestressed beams having limiting conditions a) to b), under which working load tensile stresses do not develop.

Type II are partially prestressed beams with limited tensile stresses at certain stages of loading, but ensure freedom from cracks under working load; this type is determined by limits "b" and "c". The lower limit "c" which corresponds to a zero point $W - W'$ has to be adjusted according to the kind of loading to be applied; e. g. for fatigue loading instead of taking W', only 0.5 to 0.6 W' should be deducted from W. Again, for sustained loading, not more than 0.7 W' should be considered, while for rarely occurring loading this value may be raised to 0.9 W'.

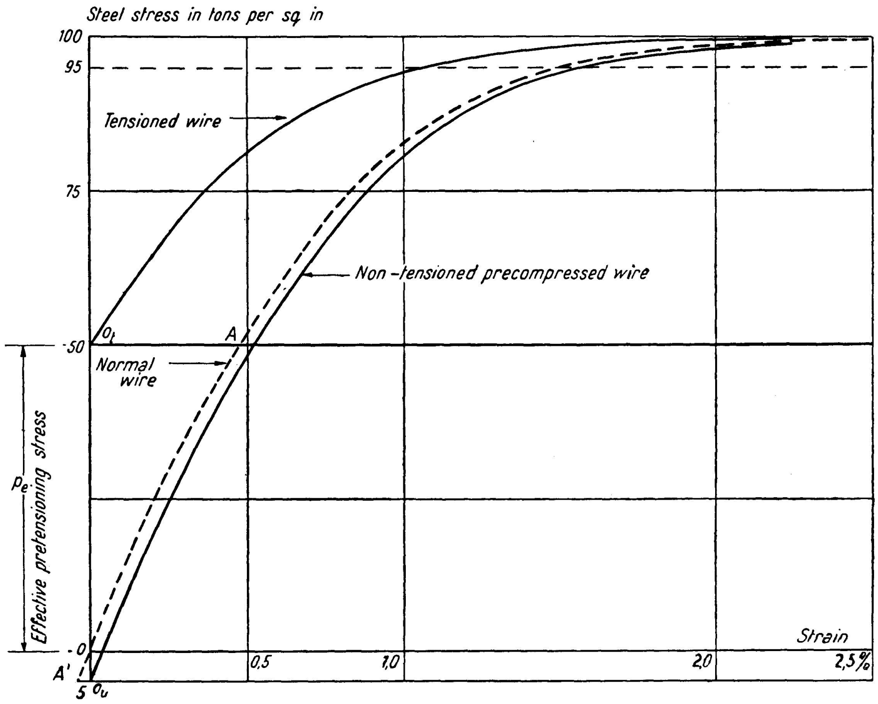


Fig. 2. Stress strain diagrams of wire 0.2 in. (5 mm) dia.

Types III and IV are also partially prestressed, Type III being moderately prestressed and IV slightly so, both types being determined by the limits "*c*" and "*d*". Type III is designed to be wholly under compression at dead weight loading and thus entirely free from cracks, whilst temporary fine cracks can develop under the rarely occurring full working load.

With Type IV beams, visible cracks may occur under ordinary loading.

Obviously beams Type II have a much greater resistance to shock than beams of Type I, the upper limit of which will suffer brittle failure, though all prestressed beams have resilience practically up to failure load, while the non-prestressed beam d) is not resilient though its shock resistance will be great.

Beams Types II to IV will give an early warning of failure in the event of overloading as seen from Fig. 1, in view of the noticeable deflections.

Since tensile stresses are permitted under working load, the required prestressing force is naturally less with partially prestressed structures, and in addition the construction depth may be reduced. Obviously the factor of safety against cracking is reduced, but this cannot be avoided if a certain resilience of the structure is desired.

If the total prestressing force is reduced, a smaller sectional area of tensioned steel is required, and in many cases this area will be insufficient to ensure the required factor of safety against failure. The author has, therefore, suggested that non-tensioned high strength wire should be added. In Fig. 2 a stress strain diagram is shown, by a dotted line, of wire 0.2 in. dia. (5 mm) having a tensile strength of 100 ton/in.^{2} and a maximum elongation of 2.5%. Assuming the initial prestress to be 67 tons per sq. in., an effective prestress

p_e of 50 tons/sq. ins. may be obtained and consequently for the tensioned wire only the part taking an origin A transferred to 0_t, at a stress of 50 tons/sq. ins. should be considered, while for the non-tensioned wire which is originally compressed and having an origin at A' transfer should be made to point 0_u. It is seen that at ultimate load conditions, with large elongations, stresses both in the tensioned and non-tensioned steel will be approximately equal.

The ratio of the required section moduli for conditions c) and b) is given by the formula [3]: $\frac{R_0 \cdot f_{ct}}{R_0 \cdot f_{ct} + f_{tw}}$, where f_{ct} is the permissible concrete compressive stress at transfer, R_0 is a reduction factor, indicating the influence of shrinkage and creep after transfer and f_{tw} is the permissible working load tensile stress; e. g. take $R_0 = 0.8$, $f_{ct} = 2000$ and $f_{tw} = 1000$ lb/in.^{2}, then the ratio becomes $\frac{1600}{2600} = 0.615$. This shows the possible reduction in section modulus due to the adoption of partial prestressing.

The author suggested partial prestressing on a broad line as early as 1940 [4] and again more in detail both in 1941 [5] and 1945 [6]. The proposal was, however, considered as an "inadequate" solution, before it was possible to prove, by static and fatigue tests, that partial prestressing is not only an economical but also a completely satisfactory solution when considering its increased resilience.

2. *Practical Aspects*

It was not until 1948 that the first application of practical prestressing by the Eastern Region of the British Railways was possible on a moderate scale. Two bridges carrying public roads over a railway were built, in which non-tensioned mild steel bars were combined with tensioned high tensile wires as the required tensile reinforcement. Full size beam tests [7], carried out in 1949 in connection with this bridge contract proved the complete co-operation of tensioned and non-tensioned wires up to failure, as indicated in Fig. 2. Based on these tests, 14 partially prestressed bridges were subsequently built in 1949—1952 in composite construction of 3 different sizes shown in Fig. 3, which will be discussed in more detail later [8, 9]. The prestressed components of most of these later bridges were cast in supersulphated metallurgical cement manufactured in Belgium.

In addition to these standardised railway overbridges, partial prestressing has also been employed in the design of roof constructions, e. g. of the Goods Shed at Bury St. Edmunds, Suffolk, part of which was built in April 1952, and this will be described later.

In all these structures Type II members (Fig. 1) were used so as to secure freedom from cracks under working load, in spite of permissible concrete tensile stresses up to 700 lb./in.^{2} under wind and snow load etc. All these structures were also designed to comply with the special condition of Type III, i. e. that they are always in compression when under normal dead load, so that any cracks which might have developed under an unexpected overloading, will close completely upon return of normal loading.

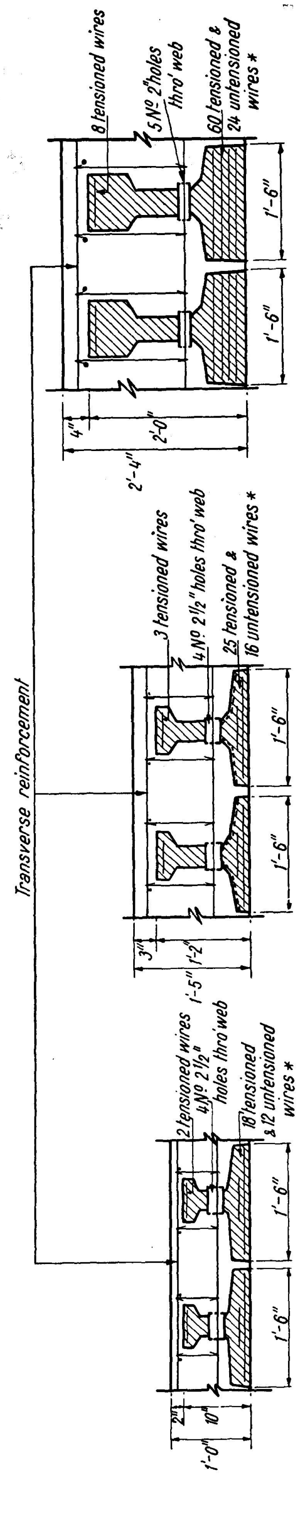
Although the advantage of partially prestressed structures (i. e. greater resilience, greater economy and reduced depth) are obvious after the test results have become known, this type of construction has, so far as can be ascertained, been used only by British Railways apart from ground supported roadwork and composite construction. In the latter type of construction the maximum concrete tensile stress normally does not occur at the outer fibre but at a plane less remote from the neutral axis inside the combined section.

It would appear that many Engineers do not appreciate the satisfactory test results, or have not yet realised the economical and technical advantages associated with partial prestressing.

3. *Full Size Tests on Partially Prestressed Concrete Members*

As described in paper [7], 3 beams, each having a span of 26′ 6″, were tested in 1949, two of which were composite beams with additional untensioned wires. The dimension of the beams corresponded to Size 2 of Fig. 3. These beams were manufactured as an addition to the first bridge contract of 1948.

The purpose of the test was to obtain data for checking tests with regard to the deflection and commencement of cracking and to investigate the cooperation of tensioned and non-tensioned wires as well as that of prestressed and added plain concrete. The full co-operation was established, resulting in complete resilience, cracks closing up and becoming entirely invisible on removal of the load.



* Number varies slightly according to span

Fig. 3. Standardised partially prestressed composite bridge slabs (cross section of 3 sizes)

Visible cracking was observed at a load corresponding to a resultant bending tensile stress in the concrete of approx. 1000 lb./in.². Consequently, checking tests were introduced as a routine control of further manufacture, as follows:

One beam of each batch manufactured on the same bed was tested to a load corresponding to a concrete tensile bending stress of 750 to 800 lb./in.². At this loading cracking was not permitted to occur and the deflection was

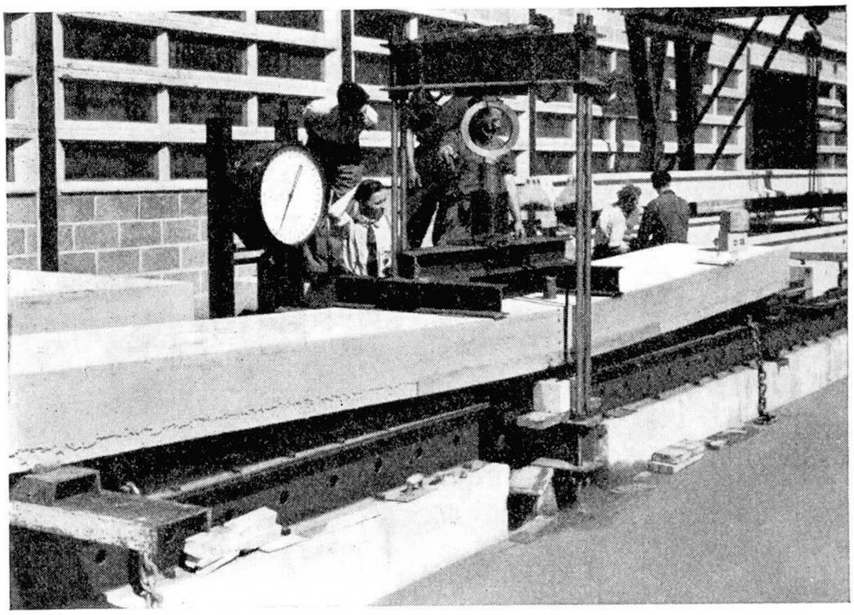


Fig. 4. Static test on partially prestressed composite slab, 1951

not to exceed a value calculated for $E = 3.75 \times 10^6$ lb./in.^{2}. Although this might at first be considered as a very severe test of concrete tensile strength in view of the close approach to the modulus of rupture, all beams tested in four bridge contracts passed the conditions; in fact, the measured deflection corresponded in most cases to E values between 5×10^6 and 6×10^6 lb./in.^{2}.

By these routine tests, good quality of the concrete can be verified more reliably than by means of cube tests. The efficiency of the prestress is checked at the same time, since the test load approaches the cracking load. One test (span 26 ft. 6 in., two point loads 1 ft. apart) is noteworthy, the load was maintained for 30 days without any cracks developing under this tensile stress of 775 lb./in.^{2}. During this time the total deflection increased from 0.768 in. to 0.856 in. after 5 hours to 0.884 in. after 1 day to 0.958 after 6 days and to 1.327 in. after 30 days, the measurements being taken daily. The nominal "E" values, corresponding to the respective central deflections, varied between 5×10^6 and 2.8×10^6 lb./in.^{2}.

In 1951 static and fatigue tests were carried out on composite slabs Fig. 4 shows one slab in a static test (carried out by Mr. Harry Stanger's Testing Institute at the Works of Atlas Stone Co. Ltd.) just before failure, when the minimum mean steel stress in the tensioned and non-tensioned wires amounted to approx. 98% of the ultimate strength. Details of these tests are described in paper [9], but some particulars about the fatigue tests carried out at Professor F. Campus' Testing Laboratory at Liège University, in conjunction with the Railway Executive Research Department, are given in the following.

The purpose of these fatigue tests was to investigate the behaviour of a partially prestressed bridge deck construction designed for concrete bending tensile stresses of 500 to 600 lb. per sq. in. under working load. Although no cracks ought to occur, even if the maximum working load were applied under fatigue conditions, it was considered possible that excessive loads might occur occasionally, resulting in the development of cracks. The slabs were first cracked by a static loading and afterwards a fatigue test was carried out in accordance with a prearranged sequence. The slabs were tested in inverted position with upward acting loads.

One slab was tested to failure after one million repetitions of loadings had been completed, when the ultimate load corresponded to 96 per cent of the steel strength.

The other slab was broken, after three million repetitions of fatigue loading had been completed, the third million being carried out in a range between nominal tensile stresses in the beams of 300 and 900 lb. per sq. in. based on forces in the jacks. Thus during this last million of repetitions there was a continuous tension in the concrete, the cracks widening four times per second, but not completely closing. During this period two outer tensioned wires fractured were exposed for $\frac{1}{2}$ in. length in slots provided for affixing gauges. Nevertheless a slightly higher failure load was obtained than with the other slab reaching approx. 100 per cent of the steel strength of 58 wires. This proves that fatigue does not affect well bonded wires in prestressed concrete up to 0.2 in. (at least of British wire of 0.2 in. dia. having satisfactory surface conditions) whether they are tensioned or untensioned, the full strength of both types being reached at failure, though the same wire failed, under fatigue conditions at approx. half the load where not bonded. Thus the ultimate resistance of under-reinforced prestressed concrete with bonded wires is not affected by fatigue.

These severe tests have also proved that if cracks have occurred in prestressed concrete and open under subsequent loading, they will close completely on removal of the load, even if the loading is repeated one million times. Thus the claim of the opposers of partial prestressing, who state that even the occasional occurrence of fine cracks is dangerous, is shown to be wrong.

II. Examples of Practical Application

1. *Standardised Composite Bridge Slabs*

The cross sections of three different standard sizes are shown in Fig. 3, Size (1) relate to short bridges of approx. 12 in. depth and 20 ft. span, while Size (2) of a depth of approx. 17 in. corresponds to a span of 30 ft. and Size (3) of a depth of 2 ft. 4 in. was used in a bridge of 48 ft. span. This design is a further development of certain comparative designs presented in the closure to paper [6] and based on a proposal by Dr. Hajnal-Konyi to combine precast prestressed and in-situ concrete. The prestressed precast beams are designed to carry the weight of the in-situ concrete and form the shuttering for it. The combined section carries the road surfacing together with the live load without the occurrence of tensile stresses under dead weight.

The precast prestressed members are designed in such a way that they can be lifted and transported even when supported near the centre. This is of great advantage for easy handling, although in this case the depth of the section is not reduced by the counteraction of the self weight to the prestress. However, this is, in any case, difficult to obtain with pre-tensioning. To increase the economy of this solution, it is important to reduce the expensive precast prestressed component to a minimum. The cost, in the Spring of 1952, of this component, may be taken as approx. £ 2. per. cu. ft. complete, placed in position, while the cost of the added concrete, on the same basis, may be taken as 5/– per cu. ft., i. e. $\frac{1}{8}$ of the prestressed component.

There is a small transverse reinforcement in the in-situ concrete which is required to ensure complete load distribution and obviates the necessity for the transverse prestressing required if precast slabs are used. When the relatively small cost of the additional in-situ reinforcement is neglected, the cost, per cubic foot, of a composite partially prestressed slab will be from 14/– to 16/4 d, if the precast component varies between $\frac{1}{4}$ and $\frac{1}{3}$ of the whole, which is much less than the cost of a precast prestressed slab. Details about some bridges are given in papers [8] and [9].

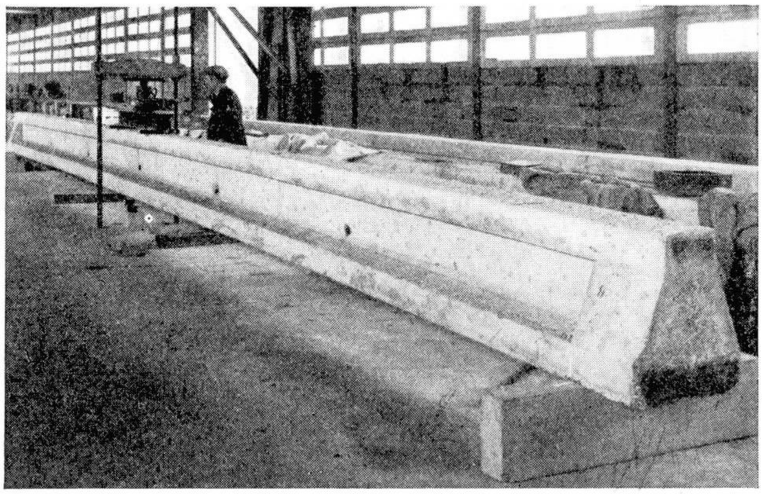


Fig. 5. Checking test bridge beam 50 ft. at Shorne 1952

In Fig. 5 is seen a checking test carried out at the Atlas Stone Company Limited on the 11th January 1952, on a beam of 48 ft. span tested with two loads of $5\frac{1}{2}$ tons each 5 ft. apart, corresponding to a tensile stress of 800 lb./sq. in. The maximum deflection corresponded to an E value of 6.5×10^6 lb./sq. in. In Fig. 6 half the number of beams have been placed in position upon a skew bridge at Silverwood, Yorkshire, while Fig. 7 shows the same bridge

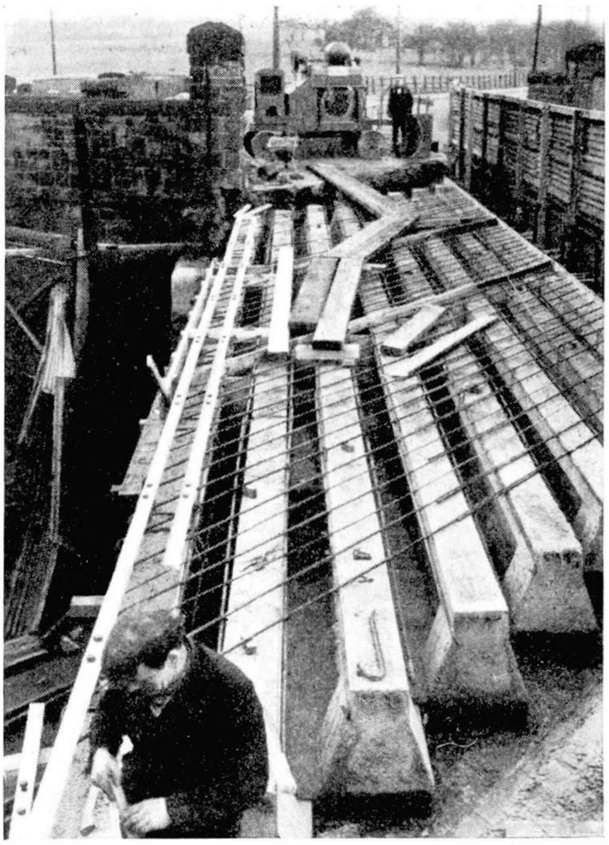


Fig. 6. Silverwood bridge, first part built 1951

Fig. 7. Silverwood bridge, completed stucture 1952

completed. At this and many other bridges Sealithor super-sulphated metallurgical cement was used. On account of this, it is possible to omit smoke boards, and thus reduce maintenance costs. Fig. 7 clearly shows the affect of smoke upon the surface, and this photograph was taken only two months after erection.

2. Roof Construction Goods Shed. Bury St. Edmunds, Suffolk

This was the first roof construction in which large partially prestressed beams were used. The main roof beams of 76 ft. 3 in. span are 20 ft. apart. The wires are post-tensioned on the Magnel-Blaton system. Eaves beams 38 ft. long with pre-tensioned wires span longitudinally between the columns to carry intermediate roof beams. Special window beams are provided at one side between the columns.

Fig. 8a shows a cross section through the centre and Fig. 8b an end elevation of the hipped roof beam, indicating the end plates of two straight cables of 16 wires and of one slightly curved cable of 24 wires, all of which are 0.2 in. dia. Very little friction occurs with this arrangement of slightly curved

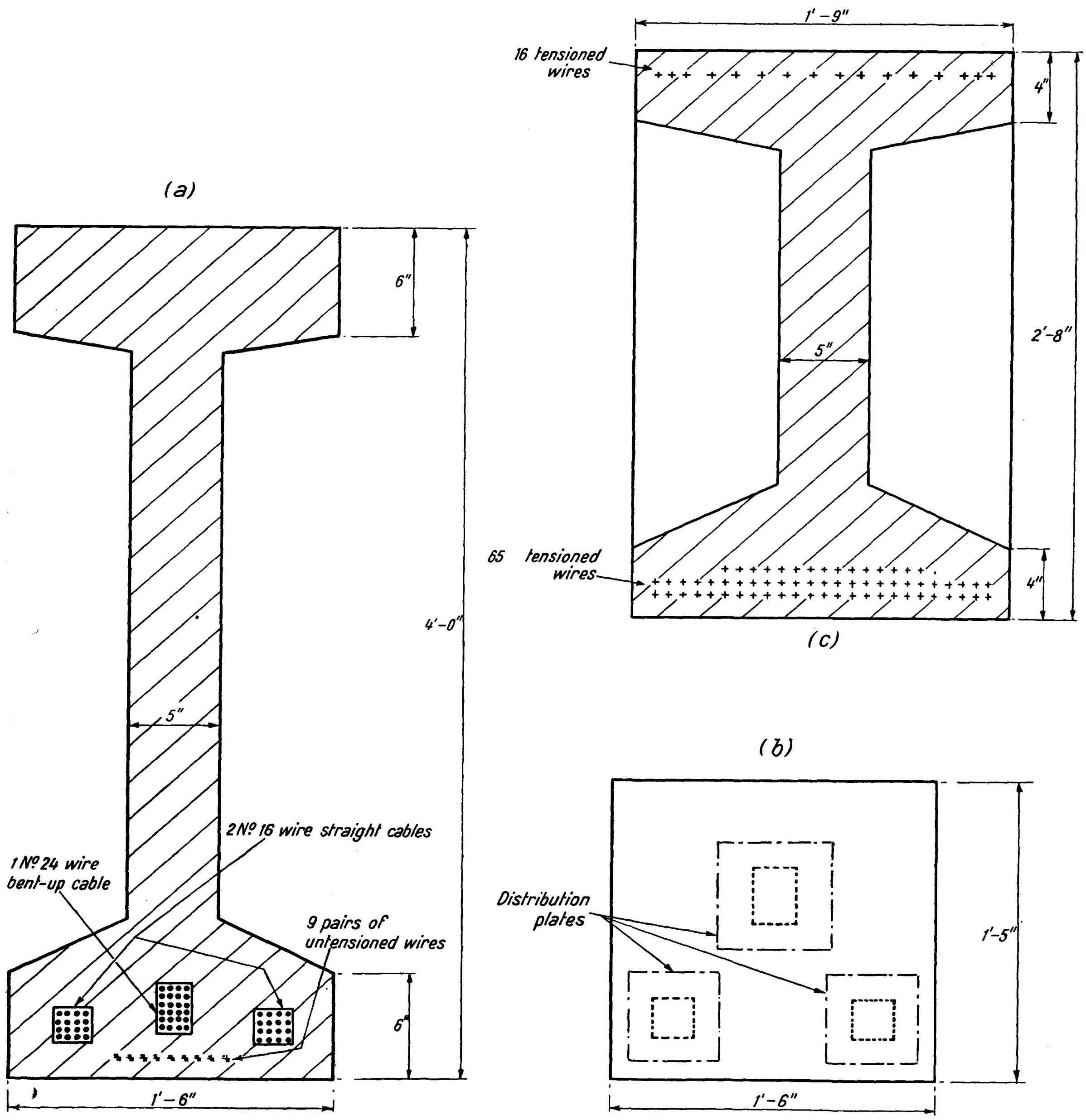


Fig. 8. Cross sections of (a) and (b) roof beam and (c) of eaves beam, Bury St. Edmunds

and straight cables. In addition, there are nine pairs of non-tensioned wires of the same diameter twisted with a pitch of 1 ft. Ribs of approximately 12 in. width are provided at the ends. The slope of the roof beams, which are 1′ 5″ deep at the ends and 4′ 0″ deep at the centre, depends upon the rise of 4° of the asbestos roofing. This sheeting is supported by prestressed purlins 6′ 3″ apart and 20 ft. long. Fig. 8c shows a cross section of an eaves beam which has a 6 in. wide stiffener rib at the centre and 12 in. wide ribs at the ends.

A checking test was carried out at the Anglian Building Products Ltd., Lendwade, on 21st April 1952, where an eaves beam was loaded with two point loads each 15 tons and 5 ft. apart upon a span of 37 ft., when the maximum deflection corresponded to an E value of 6.2×10^6 lb./in.^{2} at a concrete tensile stress of 750 lb./in.^{2}.

Such a tensile stress would occur only at an edge in the structure itself, under combined maximum vertical and horizontal loading. It is a rather rigorous condition to avoid the occurrence of any cracking under sustained loading at one edge of the beam during an exceptional loading. In consequence, the eaves beams are heavily prestressed. It might have been preferable to design them as Type III allowing fine cracks to develop under this unusual loading.

For prestressing, the 78 ft. long roof beams were supported at casting only by cross members 6 ft. apart with concrete bases. Side parts and pallets between the supports were struck after approx. 24 hours. One beam was cast in approx. $4\frac{1}{2}$ hours using external and internal vibration and three beams were cast per week. The cube strength of the concrete at 5 days was over 6,000 lb./sq. in. Fig. 9 illustrates a loading test, where 5 of the eaves beams, all of which were cast in one long bed, are used as kentledge for 2 roof beams which were jacked up to take the test load. These 5 eaves beams are of a weight of approx. 6 tons each, and the four point loads of $7\frac{1}{2}$ tons each were applied at 25 ft. centres on the 76 ft. span of the elevated roof beams.

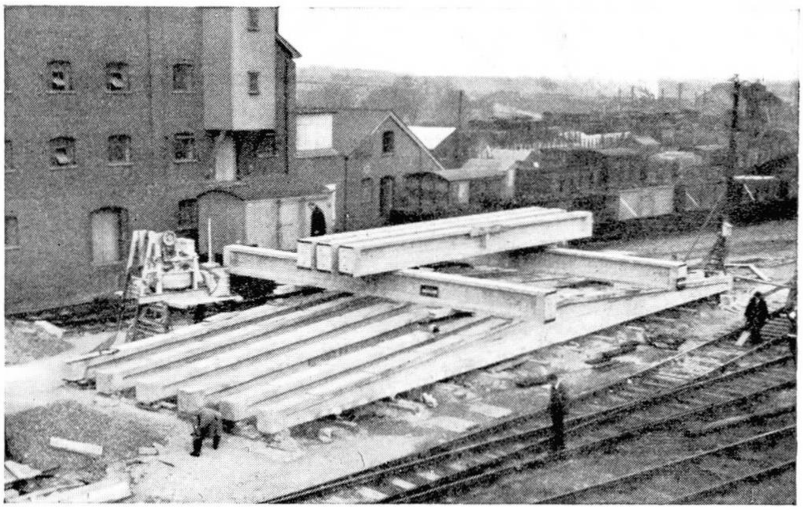


Fig. 9 Bury St. Edmunds checking test. Roof beams April 1952

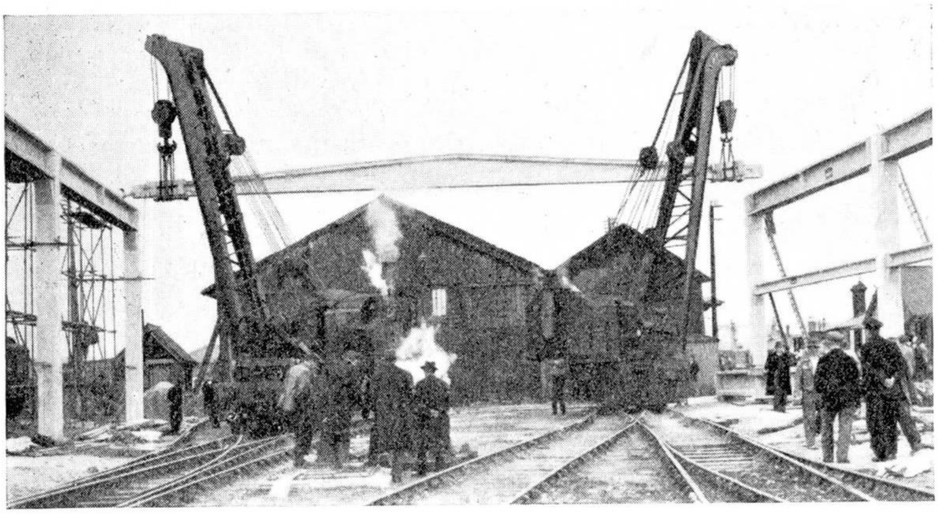


Fig. 10. Bury St. Edmunds. Placing of roof beam April 1952

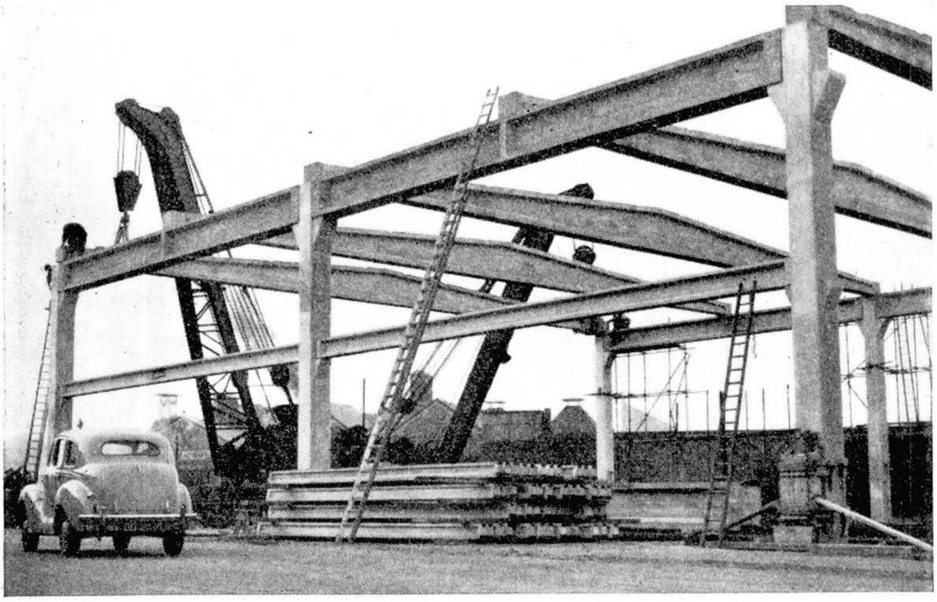


Fig. 11. Bury St. Edmunds. Prestressed beams of first part of roof

It may be noted that the maximum stress in the roof beam is not developed at the centre but near the quarter points. The test loading carried out on the 28th April was in agreement with a tensile stress of 650 lb./in.^{2} and the maximum deflection corresponded to an E value of approx. 5×10^6 lb./in.^{2}.

Fig. 10 shows placing a roof beam using two railway cranes and Fig. 11 illustrates the roof members for the first part of the shed placed in position, the weight of a roof beam being 13 tons.

The bridge work described in this paper was carried out by Messrs. Wellerman Bros. Ltd. of Sheffield, and the bridge beams for the first contract were manufactured by Messrs. Dow Mac Ltd. at Tallington, Lincs., while those of three other contracts were manufactured at the works of Atlas Stone Co. at Shorne, near Gravesend. The main Contractors of the Goods Shed, Bury St. Edmunds, are Messrs. C. R. Price, Doncaster, who made the roofing beams, while Anglian Building Products Ltd. of Lenwade, near Norwich, supplied the factory made beams and purlins. Placing of the members shown in Figs. 10 and 11 was carried out by the Railway Executive's own cranes.

The Author expresses his thanks to the Civil Engineer of the Eastern Region, Mr. J. I. Campbell, M.I.C.E., for permission to use all the particulars presented in this paper, and he is particularly grateful to the Assistant Engineer (New Works) for his kind support.

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Summary

Four types of structures are discussed and the essential features of partial prestressing are pointed out, particularly its economy, increased resilience and capacity of giving an early visible warning of overloading. Reference is made to static loading tests to failure in 1949 and 1951 and to fatigue tests carried out at Liège in 1951. Routine tests to investigate the quality of products are described and examples of practical application are given of 16 road bridges over railways of spans 20—50 ft. built as composite partially prestressed slabs. Another example relates to a roof construction at which large partially prestressed beams were used for the first time.

Zusammenfassung

Vier Typen von Konstruktionen werden erörtert und hierbei die wesentlichen Merkmale teilweise vorgespannten Betons besprochen (insbesonders Wirtschaftlichkeit, erhöhte Elastizität und Eignung, rechtzeitig eine sichtbare Warnung drohender Überbelastung zu geben). Sodann werden in den Jahren 1949 und 1951 durchgeführte statische Bruchversuche und in 1951 in Lüttich durchgeführte Dauerversuche erwähnt und der Untersuchung der Qualität dienliche Routineversuche beschrieben. Schließlich wird die praktische Anwendung von teilweiser Vorspannung auf 16 über Eisenbahnen führende Straßenbrücken von 6—15 m Spannweite gezeigt, bei welchen genormte Platten in kombinierter Bauweise verwendet wurden. Ein anderes praktisches Beispiel bezieht sich auf eine Dachkonstruktion, bei welcher teilweise vorgespannte Balken größerer Spannweite zum ersten Male verwendet wurden.

Résumé

L'auteur étudie quatre types d'ouvrages et met en évidence les caractéristiques essentielles de la précontrainte partielle et tout particulièrement l'économie, l'accroissement de la résilience et la possibilité de rendre tôt visible la surcharge. Référence est faite à des essais de mise en charge statique jusqu'à rupture, effectués en 1949 et en 1951, ainsi qu'à des essais de fatigue exécutés à Liége en 1951. L'auteur décrit des essais „de routine" ayant pour but d'étudier la qualité des matériaux et expose les exemples d'application pratique que représentent 16 ponts-routes sur des lignes de chemins de fer ayant des portées de 6 à 15 mètres et réalisés sous la forme de dalles partiellement précontraintes. Un autre exemple de précontrainte partielle est fourni par la construction d'un toit.