be made. A Paper † on the subject had been presented to The Institution by Messrs. Guthlac Wilson, M. Inst. C.E., and Henry Grace, Assoc. M. Inst. C.E.

The results given in the Paper represented only a beginning in the study of the mechanical behaviour of London clay. Additional data were being collected as opportunity permitted and the Authors hoped that engineers would contribute data which they obtained, so that a body of information could be built up.

† "The settlement of London due to Underdrainage of the London Clay." Not yet published.

# CORRESPONDENCE

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## "Relative Merits of Wire and Bar Reinforcement in Pre-stressed Concrete Beams." †

By Rhydwin Harding Evans, M.Sc., Ph.D., Assoc. M. Inst. C.E.

### Correspondence.

**Dr. K. W. Mautner** observed that the surface-area of the forty-four wires in the wire beams with a maximum initial stress of 103·2 tons per square inch, as compared with the initial stress (29 tons per square inch) of the bar beams (diameter of bar 1 inch), was 3·5 times that of the reinforcement stretched to $\frac{1}{3{\cdot}55}$ of the stretched bars. It was useful to take such a highly exaggerated example for test purposes, but that wide difference in diameter and stresses did not exist in practice. Except for small elements with small external forces no wire of less than S.W.G. 11

† Journal Inst. C.E., vol. 18 (1941–42), p. 315 (Feb. 1942).

(diameter 0·116 inch), or for very small elements S.W.G. 14 (0·080 inch) was practicable: too many wires proved troublesome. The average diameter of hard drawn wire (B.S.S. No. 236—1941) used was 0·2 inch.

In practice the maximum diameter of high-tensile (alloy) steel was 0·5 inch.

As hard-drawn wire was available (special improved steel) with 90–100 tons ultimate tensile stress and 70—80 tons 0·1 per cent. proof load, the highest initial stress might be assumed as 70 tons per square inch. For the alloy high-tensile steel a maximum initial stress of 35 tons per square inch was justified.

A comparison of those diameters and initial stresses showed that the number of wires was 3·125 times the number of bars, whilst their surface area was only

$$\frac{3{\cdot}125 \times 0{\cdot}2}{0{\cdot}5} = 1{\cdot}25 \text{ times the surface-area of the bars.}$$

In 1929, Freyssinet showed that the bond of stretched bars or wires was secured not only by adhesion, but also by wedging action, owing to the transversal swelling of the stretched reinforcement when contracting by deformation of the concrete. That swelling occurred mainly on the ends (where the transversal resistance of the concrete was least) and created small cones; therefore, special anchorage was unnecessary for bars of diameter less than $\frac{1}{5}$ inch.

In tests, of which the results had not yet been published, non-anchored wires of $\frac{1}{5}$ inch diameter broke at 100 tons per square inch without any slipping of the wires (adhesion length = distance of failure crack to end of beam = 13 inches) corresponded to a bond stress of 860 lb. per square inch.

It should be noted that so long as the cross-section remained homogeneous, the shear stresses, owing to the parabolic distribution, were much lower than those for the usual reinforced-concrete beam with first cracks.

Alloy steel bars (35 tons per square inch yield) might be practically applied only up to $\frac{1}{2}$ inch diameter. But "bar reinforcement" had been almost entirely abandoned, and wires of 0·128—0·2 inch diameter with 90—70 tons per square inch initial stress, were used.

In unbonded pre-stressed structures with large loads and spans, cables of straight-lined wires with initial stresses of 80 tons per square inch were used; they required no more space than "bar reinforcement" of 1 inch diameter, but stretched with 2·3 times the initial stress of the latter, thus avoiding the relatively far greater losses of the bars with smaller stresses by deformation, shrinkage, and creep (compare Table I, p. 326).

As small initial steel stresses entailed very high percentage losses, it was necessary to apply the highest practical initial stresses, as had been shown by Freyssinet.

The attempts carried out before Freyssinet (for example, in 1907, by Koenen, Lund, and Mandel) had failed because the losses consumed the greater part of the initial pre-stressing of 12 tons per square inch.[1]

The use of steel of 40—45,000 lb. per square inch (18—20 tons per square inch) initial stress was wrong—the pre-stressing stress would disappear almost entirely.

Certainly the working load of pre-stressed beams should not exceed that necessary to counteract the residual compressive pre-stress in the concrete, after allowing for dead weight and all losses. Therefore, stressing the concrete up to its tensile resistance with additional unstretched reinforcement as suggested by others eliminated the greater part of the advantages secured by "full pre-stressed" concrete.

With regard to bar beams without bond, the Paper did not include any record of measurements, or a comparison with conventional-type beams, as mentioned in the penultimate paragraph of the conclusions.

In beams with pre-stressed reinforcement without bond, the stretching of the reinforcement was carried out almost exclusively by using the hardened structure as abutment for the stretching device.

In those cases the loss by deformation was nil, unless stretching was carried out for one reinforcement after the other, thus leading to interactions; therefore, the stretching force of the first had to be slightly higher, and so forth. Shrinkage and creep losses were reduced, and if the same initial high-tensile stress of steel were applied (by using the cables described) the losses were smaller than in bonded wire beams.

In such "not-bonded" pre-stressed beams, the stretched reinforcement was curved and bent to low angles and a large radius (in order to avoid losses by friction), and anchored at the end, the centre of gravity of steel coinciding with that of the end section. That added useful vertical pre-compression and delayed further the appearance of the first cracks. (In that case the factor $\dfrac{-Ty^2}{K^2}$ for the strain in concrete was non-existent).

For a load $F = \dfrac{3{\cdot}95}{2}$ tons,

$A = 40$ square inches; $E_c$ supposed $= 4 \times 10^6$ lb. per square inch;

$y = 3{\cdot}5$ inch; $E_s = 30 \times 10^6$ lb. per square inch.

Dr. Mautner had found $T = 5{,}675$ lb.: or an additional strain stress of 7,300 lb. per square inch, equal to 16 per cent. of the initial stress of the "21-ton bar beam" (Table III), having an initial stress of 20 tons per square inch only.

The bending moment of $4{,}400 \times 46 = 202{,}000$ lb.-inches produced

[1] E. Freyssinet, "*Idées et voies nouvelles. Science et Industrie* (*Travaux*), Jan. 1933, chapter iv, para. 1 ("*Causes des insuccès des premières tentatives*).

in the homogeneous section stresses of $f = \pm\ 3{,}040$ lb. per square inch, which were much higher than in beams applied in practice.

For the " wire beam " before appearance of the first cracks, the additional strain stress would be $m \times 3{,}040 = 10 \cdot 2$ tons per square inch, or 14·2 per cent. of the initial stress (see Table III, p. 327, *ante*, Column II).

In practice no higher pre-stressing than 1,500 lb. per square inch would be applied, and in that case the additional strain stress would be about 7 per cent. (or up to 10 per cent.) as shown by all previous tests on the Continent and in Great Britain.

It was true that the stress increased rapidly with the first crack ; but, as with the elimination of tensile stresses, and partly of tensile shear stresses for $1 \times$ load, a safety factor against cracks of $2 - 2{\cdot}25$ had been given by large-scale tests ; a safety factor of $3{\cdot}3 - 3{\cdot}5$ against failure was usually obtained.

**The Author,** in reply, observed that the difference between the stress and the surface-area of the wire and bar reinforcement was large, but the S.W.G. 14 (0·08 inch diameter) was consistent with the size of the model beams. The use of higher ultimate strength carbon steel or nickel-chrome steel was desirable, but the limitations imposed on the supply of steel by the Iron and Steel Order compelled the Author to use steel of 40–50 tensile strength.

In maintaining the same total initial pre-stressing force a single 1-inch diameter bar was found to be necessary, as four ½-inch diameter bars would have led to greater uncertainty in the distribution of the initial pre-stressing force between the top and bottom reinforcement. That, whilst not so important in practice, was always necessary in experimental investigations. The desired comparison between low and high initial stresses was, however, achieved (see *Fig. 3*), as there remained after all losses sufficient pre-stress in the bar-reinforced beams to give an appreciable increase in strength. The series of tests made illustrated the effects of initial and final steel pre-stresses of from 0 to 193,000 lb. per square inch, the latter producing in the concrete a compressive pre-stress considerably higher than that permitted in practice. The Author agreed that cables, having an initial stress of 80 tons per square inch, would have resulted in much less loss of pre-stress due to strain, shrinkage, and creep, that being in agreement with equation (3) on p. 325, *ante.*

Dr. Mautner was mistaken in assuming that no record of measurements or a comparison with the conventional type of reinforced concrete beam had been included in the Paper, as they were shown plotted in *Figs. 3* and *6.* Tests were made on a beam subjected to a total initial pre-stressing force of 25 tons, with the bond of the tension reinforcement eliminated, the beam being identical in other respects with the 25-ton bonded pre-stressed beam. Observations were taken of the central deflexion and of the tensile and compressive strains on the concrete. The central deflexion and the compressive stress in the concrete (as calculated from the observed concrete strains) for that 25-ton unbonded pre-stressed beam were given in *Figs. 3* and *6* respectively. Those results demonstrated that the deflexion and the concrete stress of the unbonded beam were higher than those of the conventional beam. It was correct to state that the losses were reduced considerably in no-bonded pre-stressed concrete beams (where the pre-stressing was carried out after the concrete had matured), that being an advantage of the unbonded type of pre-stressed beam. The distribution of horizontal, vertical, and shear stresses in a no-bonded beam had been already investigated by the Author[1].

The factor of safety against vertical cracking of the concrete was given by the ratio:

$$\frac{\text{Final concrete compressive pre-stress} + \text{tensile strength of concrete}}{\text{Final concrete compressive pre-stress}}.$$

It was obvious that, for a factor of safety of 2·0 against cracking, the tensile strength of the concrete should have a value equal to the final concrete compressive pre-stress.

The Author agreed that when the final concrete pre-stress was about 1,000 lb. per square inch, the factor of safety against cracking could be 2·0, as the effective tensile strength or the modulus of rupture of the concrete might also be approximately 1,000 lb. per square inch. When the final concrete compressive pre-stress was of the order of 1,500 lb. per square inch, the factor of safety against cracking would become less than 2·0. In practice, however, the factor of safety against complete failure, or the load-factor, was more important, and many tests on pre-stressed concrete beams had shown that that was usually at least 3·0.

[1] "Experiments on Stress Distribution in Reinforced Concrete Beams." Structural Engineer, vol. 14, p. 124. 1936.