INVESTIGATION OF TRANSFER BOND IN PRETENSIONED PRESTRESSED CONCRETE MEMBERS BY AN ORIGINAL METHOD

by

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in the Department of CIVIL ENGINEERING

We accept this thesis as conforming to the required standard

THE UNIVERSITY OF BRITISH COLUMBIA

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ABSTRACT

A simple method is presented, suitable for the repetitive testing necessary to investigate fully the bond characteristics in the anchorage zones of pretensioned prestressed members.

Conditions in one anchorage zone of a member are simulated, the central zone of constant stress being replaced by a rigid steel frame. Loads in the wire on either side of the specimen are measured by load cells incorporating strain gauges. The experimental series comprised thirty specimens, of which five were prestressed with bright wire. Seventeen specimens were prestressed with rusted wire and the remainder were cast with rusted wire under zero prestress.

The results show the superior anchorage characteristics of rusted wire over bright wire and the exponential nature of the load pickup of the wire in the anchorage zone. A relation is suggested between end pull in at the free end and ultimate load anchored. A short description of an investigation of the relaxation behaviour of prestress wires is appended to the thesis.
ACKNOWLEDGEMENT

The author wishes to express his thanks to his adviser, Professor S.L. Lipson, for his valuable suggestions and guidance. It was a pleasure to work under his supervision. The author also expresses his indebtedness to the staff of the Civil Engineering workshop for their help, and to Mr. B. Ferman of Superior Concrete Ltd. who gave every assistance.

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June, 1962

Vancouver, British Columbia
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CHAPTER 1

INTRODUCTION

Definition of transfer bond.

Concrete members are usually reinforced in the tension zone by the inclusion of steel reinforcement. In pretensioned prestressed concrete members the steel is tensioned before pouring the concrete and then released when the latter attains sufficient strength. The steel, on release, tends to return to its original length, but is restrained by the concrete, which in turn is compressed. The transfer of force between the two materials is by means of bond at the interface. This bond is referred to as prestress transfer bond and is present from the ends of the members to the beginning of a region in which the steel tension is constant. The interval, at each end of the member, over which the transfer bond acts is called the transfer length or anchorage length.

Quantities influencing bond.

Quantities influencing bond may be divided into two categories, the adhesive and the frictional, although the distinction is largely one of convention. Adhesion is a loose term to describe an initial resistance to slip, that is, a resistance that is present only when slip is very small. Of the theories advanced to explain
it, micro-mechanical locking appears to be the most reason-
able. This postulates that the resistance to slip is given
by the sheer strength of the fine particles of concrete
that have been forced into micro-indentations in the surface
of the reinforcement. When a slip comparable with the size
of the micro-indentations takes place the adhesive resis-
tance disappears.

Frictional resistance: This may be assessed
by multiplying the radial pressure by the appropriate coeffi-
cient. Some coefficients given by Armstrong\textsuperscript{1} show the in-
fluence of grease and rust on the surface.

Mechanical locking: This is a phenomenon
similar to the micro-mechanical locking described previously
except that the indentations are deliberate and of the same
order of magnitude as the diameter of the wire.

Dilatancy: This effect, investigated by
Jenkyn\textsuperscript{2}, is a resistance to slip arising from the wedging
action of the fine particles produced after an initial slip
has taken place.

Wedge action: This is a frictional resistance
resulting from the setting up of radial strains due to
Poisson's effect in locations where the longitudinal strain
is changing.

The length of concrete section required to
anchor the force in a wire, by means of some or all of the
above effects, is a function of many interrelated variables.
The following investigation is an effort to determine the
anchorage length and the distribution of bond forces for one size of prestress wire. Two surface conditions are included.

Discussion of previous work.

Previous work on prestress transfer bond, notably by Evans and Robinson and Janney, has consisted mainly of highly instrumented tests on relatively few specimens. Evans evolved a very successful x-ray technique using platinum foil markers, which show a high x-ray absorption, embedded at intervals in the prestress wire and protruding into the concrete. Slip between the wire and concrete sheared off the foil at the interface, leaving two pieces of foil, marking the extent of slip which were readily visible on x-ray photographs taken through the specimen. The changes in length between markers, accurately measured from the x-ray photographs, permitted Evans to calculate steel strains, and hence stresses, directly. Bond stresses were obtained from the gradient of the steel stress distribution curve.

Evans' results indicated that the pickup of steel strain may be approximated, by an expression originated by Marshall

$$\epsilon_s = \epsilon_{so} (1 - e^{-4ax/d})$$

where:

- $\epsilon_s$ = true steel strain
- $\epsilon_{so}$ = maximum retained steel strain
- $a$ = a constant for the particular wire
- $x$ = distance from the free end
- $d$ = wire diameter
Furthermore the end slip or pull in of the wire may be related directly to the transmission length by the following expression

\[ L = k \cdot \frac{g_0}{\varepsilon_{is}} \]

where:

- \( g_0 \) = end slip
- \( L \) = transmission length
- \( \varepsilon_{is} \) = steel strain immediately before transfer
- \( k \) = a constant for the particular wire

Qualitative observations also indicated that the concrete strength had a large effect on transmission length and that the latter also increased with time, particularly in the case of smooth bright wires. Furthermore rusted wires were found to have superior bond characteristics to bright wire and that the characteristics of twisted strand were superior to either of these.

The experimental procedure adopted by Janney was more conventional than that of Evans, strain gauges were fixed to the single prestress wire and to the outer surface of the concrete prism surrounding the wire. Before pretensioning each wire was fitted with two SR 4 gauges. In each case one waterproof gauge was placed at the midpoint of the prism.

Twenty six SR 4 gauges were placed along the sides of the prism after moist curing was complete. Just
before the pretension was released readings were taken on all gauges. The tension on the steel was released and again all gauges were read. These data established the pretension in the steel just prior to release, the tension retained in the steel at the centre of the specimen after release, and the distribution of prestress.

The steel stress distributions were derived from strain measurements on the concrete surface. This procedure is only justified if, at all points along the prism, the stress in concrete is uniform over the whole cross section. At any point the total compression in the concrete must equal the steel tension at that point, however the concrete strain measured at the surface does not necessarily correspond to the concrete strain at the centre of the prism.

It can be seen that Janney's method is less direct than that of Evans and also retains some basic assumptions that are open to question. Janney's results may be summarized in three curves, Figures 1, 2 and 3. Some distributions of steel stresses, as derived from strains on the steel and on the surface of the concrete prisms are given in Figures 1 and 2. It is seen that the length which must be embedded in order to transmit the prestress fully to the concrete increases in proportion to the wire diameter. The length of embedment in each case is approximately one hundred diameters. The cross section of all prisms was $4 \text{ sq. ins.}$ and all steel was pretensioned to 120,000 p.s.i. Consequently the prestress force imposed upon each specimen,
FIGURE 1 - Stress Transfer Distribution for Clean and Lubricated 0.276 in. dia. Wire

FIGURE 2 - Stress Transfer Distribution for Clean and Rusted 0.162 in. dia. Wire

FIGURE 3 - Theoretical Stress Transfer Distribution for 0.276 in. dia. Wire
and the resulting concrete strain, were greater as the wire diameter increased.

Figure 1 gives the comparison of the stress transfer for a clean and rusted wire. The rusted wire developed the full transfer of prestress more quickly and nearer to the free end than did the clean wire. The comparison of clean and lubricated wires, given in Figure 2 shows a more marked spread. It seems reasonable to assume that this marked difference in behaviour between clean and lubricated wires arises chiefly from a reduction in the coefficient of friction bond between concrete and steel.

Janney attempts to justify this last assumption further by saying that of the three factors, adhesion, mechanical locking and friction, which may contribute to bond only the latter is important in the zones of slip of a member prestressed with smooth wires.

On this basis Janney made an elastic analysis of the transfer, similar to those made earlier by Mains, Evans, Marshall and others. As the tension is released and a wire starts to slip back into the concrete, the diameter increases in proportion to the reduction in tension. This swelling is resisted by the concrete surrounding the wire. In this respect the concrete is assumed to act as an elastic thick walled cylinder. Thus the wire exerts a radial pressure on the concrete and the frictional bond force is assumed to be proportional to this pressure and to the coefficient of friction between the steel and surrounding concrete.

When such assumptions are introduced into the
equations of equilibrium and compatibility an exponential relationship between wire tension and length from the free end results

\[ \log_e \frac{T_0 - T}{T_0} = -\frac{2L\varnothing}{r} \cdot \frac{\mu_s}{1 - \mu_s \mu_c} \cdot \frac{1}{n} \]

where:

- \( T_0 \) = initial wire tension
- \( T \) = tension after release
- \( L \) = length from free end
- \( \varnothing \) = coefficient of friction between steel and concrete
- \( r \) = wire radius
- \( \mu_c, \mu_s \) = Poisson ratios steel and concrete
- \( n \) = modular ratio

Such an expression was plotted by the present author for 0.276 inch diameter wires. The curves, shown in Figure 3, resemble experimental curves 1 and 2.

Janney concludes that the assumption that the transfer of stress is effected mainly by friction is valid, but points out that for the pretensions used in practice the concrete is stressed well into the plastic domain in the vicinity of the wire.

Purpose of this investigation.

It can be seen from the preceding discussion that previous workers in this field have reached substantial agreement as to the general nature of the bond and bond curve. There is also some agreement in their transmission lengths. However, the extensive instrumentation used by
these workers has prevented them carrying out the broader testing programme necessary to confirm and utilize their findings. Furthermore their results were obtained using ideal test specimens which bore only limited resemblance to materials and mixes used in construction practice.

The purpose of this investigation is to put forward a simple method for the determination of transmission length, whereby large numbers of specimens of realistic dimensions and material specifications may be rapidly tested, without the use of elaborate instrumentation.

It was recognized at the start that bond characteristics are affected by, if not dependent upon, many variables. Test series were therefore carried out with all variables but one constant throughout, in an effort to ascertain the effect of that variable alone.
CHAPTER 11

DESCRIPTION OF THE EXPERIMENTAL METHOD

The basic idea of the experimental method adopted is to cast only a single anchorage zone of a pre-stressed concrete beam, the interior zone of constant strain being replaced by a rigid frame through which the prestress wire passes. Several variations on this idea were considered, the frame had to be of fairly heavy section to ensure rigidity and minimize compressive strains. Finally it was decided to use 7 in. by 2 in. channels. The frame is shown in Figure 4.

The two end diaphragms and the centre diaphragm were also short lengths of 7 in. by 2 in. channel, welded to the longitudinal channels. Stiffeners were welded inside the web, at the third points of these diaphragms. A plywood base was fitted to carry the wooden forms, and wooden skids were attached to ease movement of the frame when necessary.

Prestressing a wire demanded a method which, in addition to a capacity of 20 kips, possessed a sensitivity of a few pounds and minimum creep characteristics under load. Initially a hydraulic system was designed with collars running on a threaded ram. However later experience proved that the more simple screw jack arrangement shown in Figure 4 was
fully satisfactory. The jack was turned by a lever about four feet long and ran on a specially machined plate, bolted to the diaphragm.

A robust, simple, but highly sensitive method was needed to measure tension and changes in tension in the wire. The conditions encountered in pouring concrete meant that any apparatus used had to withstand moisture, possibly steam curing and also heavy vibration. "Load cells" made by the Baldwin Lima Hamilton Company were claimed to withstand these conditions. They consist of a round solid steel rod carrying strain gauges. This rod, the dummy gauge and necessary circuitry are sealed inside a cylindrical steel case. Facilities are provided for loading in tension or compression.

In practice the load cells were altogether satisfactory in both degree of accuracy and robustness. Calibration showed that the load strain reading characteristics are very nearly linear up to full load. Readings remained very consistent through seven months of testing and zero drift was negligible.

Since, in general, the stress in the wire on either side of the specimen, see Figure 4, was not the same, it was necessary to use two load cells. They were each connected to the prestress wire by commercial wire grips, housed in screw couplings. Similar screw couplings at the other end of each load cell housed grips for the wire strand used to connect the cells to the screw jacks.
Yoke Prevents Rotation of Upper Part of Jack

---

**FIGURE 4: DETAILS OF TEST FRAME**

End View

Plan
The two load cells frequently had to be read in quick succession on the single strain indicator. This necessitated a switching unit of some kind. Commercial knife switches proved to have a variation of contact resistance of the same order as the change in gauge resistance. This problem was solved by mounting four two-way mercury switches in a watertight plexiglass box. The results obtained with this unit showed no variation due to contact resistance.

The wires used in the tests were normal commercial prestress wires of Japanese manufacture. Two surface conditions were tested, clean bright wire as delivered from the maker and wire slightly rusted after outside storage. The latter condition is that normally used in practice, some manufacturers of prestressed concrete products even induce rusting deliberately by exposing wire to the weather and spraying with water. Some creep of the wire at high tension had been anticipated, on testing however, some unusual characteristics emerged. These are fully covered in Appendix 3. Creep was minimized in the test series by applying a five percent overstress for about one minute during prestressing. Five hours were allowed between prestressing and pouring the concrete to permit creep, which would otherwise have had a harmful effect on bond.

**Mix Design.**

In an effort to conform as closely as possible
to commercial practice, as stated in Chapter 1, it was decided to use the same mix as Superior Concrete Ltd., who at that time were producing pretensioned prestressed beams. Unfortunately the mix proved too stiff for the small drum mixer available. Only minor modifications to the mix and mixer however, were necessary to produce a realistic mix of the required strength. The mix proportions and aggregate specifications are shown in Appendix 2. Vibration was essential for such a dense mix and restricted section.

It was originally thought that steam curing would be necessary to permit testing to be carried out at about 48 hours after pouring. In fact the first specimen was steam cured at the plant of Superior Concrete Ltd. The cylinder strength attained at 24 hours after steam curing was in excess of 6,200 p.s.i. Laboratory tests on the modified mix yielded cylinder strengths in excess of 6,100 p.s.i. after 48 hours of moist curing. On this evidence it was decided that steam curing was not essential to attain the strengths required (i.e. 4,500 to 5,000 p.s.i.) at 48 hours.

Experimental Series.

Thirty experiments were carried out over a period of six months. After a few initial trials an experimental cycle was evolved which was used for twenty five of the experiments. The cycle covered 47 hours and enabled two specimens to be conveniently cast in one week. The wire was tensioned between the previously oiled forms and then left
for five hours, in which time the majority of the primary creep took place. The weighing and batching were started after four hours, in time to pour the specimen and control cylinder at time five hours. These were immediately covered with damp hessian, which was sprayed continuously for about forty hours. The cylinder was temporarily removed for capping at about five hours after pouring. At forty-two hours after pouring the forms were stripped and dial gauges were fixed in the vicinity of the ends of the specimen to record any slip in the wire. The gauges were attached by magnets to the frame and bore against collars, rigidly attached to the wire.

Initial readings were taken of the gauges and load in the wire on each side of the specimen. The load in the wire on the free side of the specimen was reduced by increments, both gauges and cells being read at each increment.

If the specimen was of sufficient length to anchor the whole prestress then the load in the wire could be reduced to zero at the free end, without substantially reducing the load in the wire at the far side of the specimen. In this case the load on the tensioned side of the specimen was increased until bond failure occurred. Ideally the length of specimen could be reduced until any small increase of this sort would cause bond failure. The length of the specimen would then be equal to the anchorage length for the particular wire, concrete and prestress force used. Experiments were carried out on 0.276 in. diameter wire
under two loading conditions and two surface conditions. The cross-section of the specimen was maintained at $\frac{1}{4}$ in. by $\frac{1}{4}$ in., with a single centrally positioned prestress wire. Specimen lengths varied from 6 inches to 47 inches.
The experimental series was designed to evaluate the effect of a single variable at a time. To this end conditions were maintained constant for each test and change was made only in the single variable under investigation. Bright wire, as delivered from the manufacturer, was used in the initial tests, but unfortunately further supplies could not be obtained and the remaining tests were carried out with rusted wire. This latter condition was exactly as used by the local manufacturers of prestressed products. None of the wire surfaces were cleaned since this is rarely done in practice and would lead to artificial results.

The first five experiments comprised specimens of varying length, prestressed with bright wire. Seventeen of the remaining twenty-five tests were specimens of various lengths, prestressed with rusted wire. The rest of the specimens were cast with rusted wire under zero prestress.

The results are tabulated in Table 1. A six inch diameter control cylinder was cast with every batch and cured under the same conditions as the specimen. The cylinder was crushed, at the time of testing the specimen, at a rate of 0.05 inches per minute in the two hundred ton Baldwin
testing machine. The cylinder crushing strengths are tabulated in Column 2 of Table 1. Columns 3 to 6 are self explanatory. Column 7 shows the maximum recorded difference in load, as indicated by the load cells, in the wire on either side of the specimen. Column 8 shows the wire movement at the free end immediately before first slip. This is the time when the maximum load, tabulated in Column 7, is sustained by the specimen. The wire movement at the free end at this stage represents the "pull in" or "end slip on release of the pretension" for the anchorage of the load tabulated in Column 7. Column 8 is incomplete because the dial gauges were removed at loads in excess of 11,000 pounds to protect them from damage in sudden failures. Column 9 shows the load remaining in the wire on the "tight" end of the specimen, when the load in the wire at the "free" end has been reduced to zero. If zero slip has occurred at detension, then the tension remaining on the "tight" side, as indicated in Column 9, is about 9,000 lbs. A lower figure than this indicates that some slip has already taken place. Column 9 has no meaning for the untensioned specimens since they were only intended to furnish comparative data on the force anchored by pretensioned and untensioned wires. Column 10 shows the wire movement recorded on the dial gauge at the "free" end when the wire tension at that end has been reduced to zero. Column 11 shows the wire movement at the tensioned end for the same condition. Column 12 notes the type of failure.
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<th>Cylinder Strength (p.s.i.)</th>
<th>Length (inches)</th>
<th>Time of Curing (hours)</th>
<th>Surface Conditions</th>
<th>Load Conditions</th>
<th>Maximum Load Anchored at Load Pull in at Maximum Load Ins. x $10^{-3}$</th>
<th>Free End Load at Detension Ins. x $10^{-3}$</th>
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<td>&quot;</td>
<td>10,000</td>
<td>9,200</td>
<td>Slow</td>
</tr>
<tr>
<td>21</td>
<td>3,220</td>
<td>18</td>
<td>&quot;</td>
<td>&quot;</td>
<td>&quot;</td>
<td>8,300</td>
<td>9,100</td>
<td>Slow</td>
</tr>
<tr>
<td>22</td>
<td>4,350</td>
<td>19</td>
<td>&quot;</td>
<td>&quot;</td>
<td>&quot;</td>
<td>8,300</td>
<td>9,100</td>
<td>Slow</td>
</tr>
<tr>
<td>23</td>
<td>4,350</td>
<td>19</td>
<td>&quot;</td>
<td>&quot;</td>
<td>&quot;</td>
<td>8,300</td>
<td>9,100</td>
<td>Slow</td>
</tr>
<tr>
<td>24</td>
<td>3,960</td>
<td>6</td>
<td>&quot;</td>
<td>Tension</td>
<td>&quot;</td>
<td>3,850</td>
<td>101.7</td>
<td>Sudden</td>
</tr>
<tr>
<td>25</td>
<td>3,960</td>
<td>6</td>
<td>&quot;</td>
<td>Un tension</td>
<td>&quot;</td>
<td>2,950</td>
<td>1,650</td>
<td>Sudden</td>
</tr>
<tr>
<td>26</td>
<td>3,960</td>
<td>11.5</td>
<td>42</td>
<td>&quot;</td>
<td>&quot;</td>
<td>3,850</td>
<td>9,100</td>
<td>Sudden</td>
</tr>
<tr>
<td>27</td>
<td>3,880</td>
<td>14</td>
<td>&quot;</td>
<td>Tension</td>
<td>&quot;</td>
<td>10,150</td>
<td>91.5</td>
<td>Sudden</td>
</tr>
<tr>
<td>28</td>
<td>3,880</td>
<td>15</td>
<td>&quot;</td>
<td>Un tension</td>
<td>&quot;</td>
<td>10,450</td>
<td>8,850</td>
<td>Sudden</td>
</tr>
<tr>
<td>29</td>
<td>3,890</td>
<td>21</td>
<td>&quot;</td>
<td>Tension</td>
<td>&quot;</td>
<td>11,900</td>
<td>8,850</td>
<td>Sudden</td>
</tr>
<tr>
<td>30</td>
<td>2,780</td>
<td>11</td>
<td>&quot;</td>
<td>&quot;</td>
<td>&quot;</td>
<td>5,750</td>
<td>&quot;</td>
<td>Sudden</td>
</tr>
</tbody>
</table>

Specimen 30 was not instrumented, the concrete strength was too low.
FIGURE 5: Load Transfer Distribution for Smooth 0.276 in. dia. Wire

FIGURE 6: Load Transfer Distribution for Rusted 0.276 in. dia. Wire.

FIGURE 7: Load Transfer Distribution for Untensioned Rusted 0.276 in. dia. Wire.
The results are shown graphically in Figures 5, 6 and 7, which are curves of load anchored, versus length from the free end for particular wire conditions. Where slip took place before the load in the wire at the free end had been reduced to zero, the maximum difference anchored was plotted. In the absence of slip the load anchored after the tension at the "free end" had been reduced to zero was plotted.

Observations from experimental results.

Comparisons of Figures 5, 6 and 7 shows that bright wire attains the same anchored load in a considerably longer length than rusted wire. It will also be noticed from Table 1 that no sudden failures occured with bright wire, the failure was always of the slow type with the wire sliding through the concrete at steadily decreasing load. There is insufficient data in the short specimen region for bright wires to indicate whether the build up of load there differs from the build up in the case of rusty wires. Comparison of Figures 6 and 7 shows that a rusted tensioned wire anchors force faster than an identical untensioned wire. The difference is particularly marked in the 6 inch specimens. The pretensioned specimen anchors a maximum load of 3,850 lbs. while the untensioned specimen only anchors 2,550 lbs.

The force anchored versus length plot for rusty untensioned wire is substantially linear. The same
plot for pretensioned rusted wire would be more nearly exponential. A theoretical curve, given below and derived in Appendix 1, is shown against the experimental points in Figure 6.

\[
\log_e \frac{T_o - T}{T_o} = -2L\phi \cdot \frac{\mu_s}{1 - \mu_s \left(1 - \mu_c\right)n}
\]

where:

- \(T_o\) = initial wire tension
- \(T\) = tension after release
- \(L\) = length from free end
- \(\phi\) = coefficient of friction between steel and concrete
- \(\mu_s\), \(\mu_c\) = Poisson ratios steel and concrete
- \(r\) = wire radius
- \(n\) = modular ratio

The assumed values used in Figure 6 were:

\[
\mu_s = 0.30 \quad n = 10 \\
\mu_c = 0.15 \quad \phi = 0.4
\]

The concrete cylinder strengths show a steady drop throughout the experimental series. The same mix was used for specimens 2 to 30 and identical curing conditions were used for specimens 6 to 30. Any small variations in experimental technique would, it is thought, give rise to a more random distribution of concrete strengths. The definite downward trend can only be attributed to deterioration of the cement under storage in the laboratory.

The results as a whole show some scatter when
plotted. This is not unusual in the testing of concrete, where relations are less exact than in completely homogeneous materials. This scatter was anticipated and every effort was made to keep conditions perfectly constant. The mixing and curing processes were standardized after the initial trials and remained unchanged throughout the series. Vibration was originally continued until the mix appeared to have reached maximum density, late in the series a fixed time of vibration was used.

The load in the wire retained after release of the pretension is shown in Column 9 of Table 1. It will be observed that this load only attains about ninety percent of the initial prestress of 10,000 lbs., even when the prestress is apparently fully anchored. The reduction is partly due to relaxation of the wire, (see Appendix 2), but mainly due to elastic flexure of the diaphragm under the compressive prestress load in the concrete. This deflection is analogous to the elastic shortening of the central zone of a prestressed concrete member, which causes a similar reduction in retained load in the prestress wire. The magnitude of the reduction in the apparatus used was quite large, but the high initial stress used led to a retained stress, after detension, of 65 percent of ultimate. Use of a stronger section for the diaphragm would raise this figure further.

Conclusions.

The close agreement between the theoretical curve and observed data, shown in Figure 6, indicates that
the assumptions made in the derivation of the former are valid. The derivation is shown in Appendix 1, it assumes that the bond is due to friction, the Poisson effect in the wire is included. Calculation by this elastic theory of the stresses in the vicinity of the wire-concrete interface show that plastic conditions prevail. This would reduce the radial and hence the friction forces acting. However the elastic theory also takes no account of the axial stress in the concrete, this latter would tend to increase the radial pressure due to Poisson's effect in the concrete. The foregoing conclusions are strengthened by the difference of behaviour of tensioned and untensioned specimens. The former are able to anchor a higher load in the short specimens, although the anchored loads become equal at a specimen length of twenty inches or more. This extra anchorage capacity is due to the approximately constant contribution of the Poisson effect, not present in the untensioned specimens. This effect is relatively large in the short specimens, decreasing in relative importance as the length increases.

The fact that rust improves the bond characteristics agrees with the foregoing, since the coefficient of friction between rusted wire and concrete is higher than that between bright wire and concrete.

The modes of failure, tabulated in column 12 of Table 1, show that no bright wires were subject to sudden failure, but that ten out of twelve of the rusted wires anchoring the highest ultimate loads failed suddenly. Also the specimens exhibiting unusually high ultimate anchored
loads, notably specimens 18, 27 and 28, all failed suddenly. This indicates that there is sometimes present in the bond between rusted wire and concrete an additional adhesive component, which contributes to anchorage until some micro-slip occurs. The adhesion is then broken. Friction alone is unable to anchor the load and rapid bond failure takes place without warning.

The confirmation and expansion of earlier ideas of anchorage show that this simple method of testing can indeed provide the repetitive test data necessary before the laws governing bond can become accepted in practice.
APPENDIX 1

THE THEORY OF BOND ANCHORAGE

Notation:

\begin{align*}
d & = \text{wire diameter} \\
r & = \text{wire radius} \\
l & = \text{distance from free end} \\
f_s & = \text{tensile stress in wire at any point} \\
f_{se} & = \text{initial tensile stress in wire} \\
\mu_s & = \text{Poisson's ratio steel} \\
\mu_c & = \text{Poisson's ratio concrete} \\
E_s & = \text{modulus of elasticity steel} \\
E_c & = \text{modulus of elasticity concrete} \\
\sigma_r & = \text{radial stress at interface} \\
\sigma_t & = \text{tangential stress at interface} \\
\phi & = \text{coefficient of friction, steel to concrete} \\
U & = \text{bond stress} \\
T_0 & = \text{initial wire tension} \\
T & = \text{wire tension after release}
\end{align*}

If a wire is free to expand, a reduction in tension at any point from \( f_{se} \) to \( f_s \) will cause an increase of radius in the wire.

\[
\Delta r_s = r \frac{\mu_s}{E_s} (f_{se} - f_s)
\]
Thick walled cylinder theory gives the radial deformation in the concrete:

\[ \Delta r_c = -r \sigma_r \left(1 - \mu_c \right) \frac{E_c}{E} \]

for small wire diameters.

Similarly:

\[ \Delta r_s = r \sigma_r \left(1 - \mu_s \right) \frac{E_s}{E} \]

and

\[ \Delta r_s - \Delta r_c = r \sigma_s \left( f_{se} - f_s \right) \frac{E_s}{E} \]

or

\[ r \mu_s \left( f_{se} - f_s \right) = r \sigma_r \left( \frac{1 - \mu_s}{E_s} \frac{1 - \mu_c}{E_c} \right) \]

hence:

\[ \sigma_r = \frac{\mu_s \left( f_{se} - f_s \right)}{1 - \mu_s \left( 1 - \mu_c \right) E_s E_c} \]

The bond stress at any point is equal to the slope of the stress transfer curve multiplied by \( r/2 \)

\[ U = \frac{df_s}{dL} \cdot \frac{r}{2} \]

If the load is entirely transmitted by friction:

\[ U = \phi \sigma_r \]

or

\[ dL = \frac{r}{2} \frac{df_s}{\phi \sigma_r} \]
Substituting for $\sigma_r$, integrating and using the boundary condition $f_s = 0$ at $L = 0$,

$$
\log_e \frac{f_{se} - f_s}{f_{se}} = -\frac{2 \varrho \mu_s L}{r \left[ 1 - \mu_s \frac{1}{(1 + \mu_c)^E} \right] E_s E_c}
$$

or,

$$
\log_e \frac{T_0 - T}{T_0} = -\frac{2 \varrho \mu_s L}{r \left[ 1 - \mu_s \frac{1}{(1 + \mu_c)^E} \right] E_s E_c}
$$
APPENDIX 2

SPECIFICATIONS OF MATERIALS AND APPARATUS USED

Wire:

The wire supplied was specified as 0.276 in. diameter and conforming to ASTM specifications A 421-59T Type WA.

<table>
<thead>
<tr>
<th>Specification</th>
<th>Strain Gauges</th>
<th>Extensometer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Breaking load - kip</td>
<td>14.57</td>
<td>14.39</td>
</tr>
<tr>
<td>Tensile strength - k.s.i.</td>
<td>244</td>
<td>240</td>
</tr>
<tr>
<td>Yield - k.s.i.</td>
<td>218</td>
<td>206</td>
</tr>
<tr>
<td>Initial tangent modulus - k.s.i.</td>
<td>29,000</td>
<td>27,700</td>
</tr>
</tbody>
</table>

Tensile tests were carried out on two specimens of wire, one carrying a Cambridge extensometer and the other two axially mounted etched foil strain gauges. The measured specifications were as follows:

The stress strain curve derived from the readings of one strain gauge is shown in Figure 8.

The 100 ton Olsen testing machine was used for these tests.
Aggregate:

The results of a sieve analysis of the aggregate is shown below:

<table>
<thead>
<tr>
<th>Sieve No.</th>
<th>1&quot; Aggregate</th>
<th>3/8&quot; Aggregate</th>
<th>Sand</th>
<th>Combined</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&quot;</td>
<td>1.0</td>
<td></td>
<td></td>
<td>0.4</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>62.6</td>
<td>0</td>
<td></td>
<td>25.6</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>88.4</td>
<td>1.3</td>
<td></td>
<td>36.4</td>
</tr>
<tr>
<td>No. 4</td>
<td>98.8</td>
<td>92.9</td>
<td>0</td>
<td>56.9</td>
</tr>
<tr>
<td>No. 8</td>
<td>100.0</td>
<td>98.3</td>
<td>12.8</td>
<td>63.5</td>
</tr>
<tr>
<td>No. 14</td>
<td>99.2</td>
<td></td>
<td>26.5</td>
<td>69.4</td>
</tr>
<tr>
<td>No. 30</td>
<td>99.8</td>
<td>57.7</td>
<td></td>
<td>82.5</td>
</tr>
<tr>
<td>No. 50</td>
<td>100.0</td>
<td>80.7</td>
<td></td>
<td>92.0</td>
</tr>
<tr>
<td>No. 100</td>
<td></td>
<td></td>
<td>94.9</td>
<td>97.9</td>
</tr>
<tr>
<td>No. 200</td>
<td></td>
<td></td>
<td>99.0</td>
<td>99.6</td>
</tr>
</tbody>
</table>

The combination of aggregates used in the mix is shown in the right hand column. The gradings are plotted in Figure 9, the combined gradings being shown as dotted. The aggregate cement ratio used in the mix was 4.11 and the water cement ratio was 0.43. Type 3 cement was used throughout.
FIGURE 8: Load-Strain Curve for 0.276 in. dia. Wire

FIGURE 9: Individual and Combined Aggregate Gradings
APPENDIX 3

OBSERVATIONS ON THE RELAXATION CHARACTERISTICS OF PRESTRESS WIRE

Introduction:

Preliminary tests to determine the optimum mixing and curing conditions included one specimen which was cast and steam cured at a commercial plant. Immediately prior to detension, when the specimen was thoroughly cooled, it was noticed that the wire had lost about 10% of its pretension. Some relaxation was anticipated under the initial pretension, but a figure of 5% is the generally accepted value. It was considered possible that heating caused an increase of relaxation and to clarify this point a few experiments were carried out in which tensioned wires were surrounded by a water bath. These initial tests partially confirmed the idea that the amount and rate of relaxation were affected by temperature. A further eight tests were then conducted using a thermostatically controlled oil bath. The results of these tests and the conclusions drawn from them are included in this appendix.

Theory of relaxation:

Relaxation is the decrease of stress in a material held at constant strain. The problems posed by relaxation are by no means recent. The development of the
FIGURE 10: Relaxation Curves for 0.276 in. dia. Wire
whole technique of prestressed concrete was hampered by the failure of the contemporary stressing steel to maintain the necessary stress at constant strain. The advent of cold drawn, heat treated high tensile steels has reduced, but not eliminated, the problem.

The stress time or relaxation curve for the high tensile steel wire used for prestressing the specimens is shown in Figure 10.

It can be seen that practically all of the loss is attained in the first twenty hours after tensioning. This loss can be reduced if the wire is subjected to a small plastic extension. This extension effectively raises the proportional limit of the wire. The technique is widely used in practice. The wire is given a small overstress at tensioning, which is maintained for a few minutes before being reduced to the required initial tension. The relaxation curve for a wire which was subject to an initial overstress of 5% is shown in Figure 10. It can be seen that the stress loss is considerably reduced.

Efforts have been made to formulate expressions connecting the variables governing relaxation. These expressions are somewhat empirical and moreover are usually appropriate to conditions of high temperatures and stresses that are low by comparison with those used in prestressing. These are the conditions where creep normally becomes a problem.

It is accepted that, at least in the initial
stages, loss of stress varies exponentially with time. An expression of the form:

\[
\frac{\sigma}{E} - \epsilon_C = \frac{\sigma_0}{E} = \text{constant}
\]

where:

- \( \sigma \) = stress in wire
- \( E \) = Young's modulus for wire
- \( \epsilon_C \) = creep strain
- \( \sigma_0 \) = initial stress in wire.

Now if \( \epsilon_C = K t^m (e^{\sigma/s} - 1) \), as suggested by Soderberg\(^7\), where \( K, m, \) and \( s \) are constants and \( e \) is the base of natural logarithms, then the expression:

\[
\frac{\sigma - \sigma_0}{E} = -K t^m (e^{\sigma/s} - 1)
\]

predicts the stress at time \( t \) in a specimen under steady temperature conditions, the constants being adjusted to suit the particular conditions.

It is generally accepted that the rate of creep increases in some manner with temperature, but little work has been done in this field.

**Experimental series:**

Experience with the steam cured specimen suggested that at the high working stress the rate and amount of creep may be sensitive to heating. Eight experiments were made in which identical wires were stressed and then subjected to various temperatures, times of application of
heat and durations of heating. The initial tension was 10 kips in all cases except specimen 1 where the initial tension was 9.85 kips. The results are shown graphically in Figure 11. All specimens show a steady logarithmic rate of relaxation at constant temperature. The portions of the curves in red ink indicate the time for which the wire was maintained at the temperature shown.

**Experimental observations:**

Specimens 1 and 2 did not receive any initial overstress and consequently show, at any time prior to heating, a greater creep loss than specimens 4, 5 and 8, which did receive initial overstress. The disparity between curves 1 and 2 is due to the low initial tension of specimen 1, as noted in the previous paragraph. Specimens 4, 5, 6, 7 and 8 all received a 5% initial overstress and were then heated at 220°F for varying periods at various times, as shown in Figure 11. Specimens 1 and 2 were heated at successively higher temperatures with periods of cooling in between. The constant temperature regions of all curves show a remarkable linearity indicating that the relaxation time relation is primarily exponential, at least in the period (0 to 100 hours) of primary creep. The variation between specimens of the total amount of tension lost through creep is small. There is a substantially constant loss of 850 lb. at 100 hours. No increase in relaxation is caused by increase in duration of heating, the loss at a particular temperature takes place very quickly. Specimens 1 and 2 show that the
FIGURE 11. RELAXATION CURVES FOR HEATED WIRES
amount of creep is proportional to the curing temperature. Comparison of specimens 4 and 6 shows that the time after release at which heating occurs has no effect on the loss due to that heating.

Conclusions:

Normally the rate of primary creep in tensioned prestress wires is exponential, the total creep loss at 100 hours after tensioning is about 4 to 5% of the initial pretension. The application of the sort of temperatures encountered in steam curing, for one hour or more, causes a sharp drop in tension, amounting to 4 or 5% of the initial pretension. This loss is not recovered. The application of this heat at any stage of primary creep will have the same result. The amount of relaxation increases in some manner with temperature, i.e. curing at 210°F will induce a greater loss than curing at 180°F. The losses due to heating occur in addition to the normal creep loss, which seems to remain unaffected, although these results do not indicate whether ultimately the total creeps, with or without heating, are equal.

The present author was unable to find any references to previous work in this field. The losses may be a peculiarity of the particular steel used, but they attain significant proportions and warrant consideration in design.
BIBLIOGRAPHY:


