Prestress Losses in In-Service Bridge Beams and Refinement of Prediction Method

by

Ti Huang

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Evaluation of Prestress Loss Characteristics of In-Service Bridge Beams

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Evaluation of Prestress Loss Characteristics of In-Service Bridge Beams


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Prepared in cooperation with the Pennsylvania Department of Transportation and the U. S. Department of Transportation, Federal Highway Administration. The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Pennsylvania Department of Transportation, the U. S. Department of Transportation, Federal Highway Administration, or the Reinforced Concrete Research Council. This report does not constitute a standard, specification or regulation.
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For the highly prestressed members used in this study, prestress losses were found to be insensitive to the characteristics of the prestressing steel, whether stress-relieved or stabilized. The outdoor specimens showed slightly higher shrinkage and creep strains than their indoor counterparts, however, the differences were not large. The long term concrete strains predicted by the existing procedure agreed reasonably well with the field measured data. A refinement in the estimation of creep strain due to applied loads resulted in improvement agreement. Based on this refinement, prestress may be treated as remaining constant over a short period of time after application of external loads.
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABSTRACT</td>
<td>vi</td>
</tr>
<tr>
<td>1. INTRODUCTION</td>
<td></td>
</tr>
<tr>
<td>1.1 Background</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Scope and Objectives</td>
<td>2</td>
</tr>
<tr>
<td>1.3 Summary of Project Work</td>
<td>3</td>
</tr>
<tr>
<td>1.4 Definitions</td>
<td>6</td>
</tr>
<tr>
<td>1.5 Units</td>
<td>7</td>
</tr>
<tr>
<td>2. EXPERIMENTAL BRIDGE</td>
<td>8</td>
</tr>
<tr>
<td>2.1 General Description</td>
<td>8</td>
</tr>
<tr>
<td>2.2 Design, Fabrication and Erection</td>
<td>9</td>
</tr>
<tr>
<td>2.3 Instrumentation and Testing</td>
<td>11</td>
</tr>
<tr>
<td>2.4 Material Properties</td>
<td>11</td>
</tr>
<tr>
<td>3. ANALYSIS OF EXPERIMENTAL RESULTS</td>
<td>13</td>
</tr>
<tr>
<td>3.1 Short-Term Observations</td>
<td>13</td>
</tr>
<tr>
<td>3.2 Long Term Observations</td>
<td>14</td>
</tr>
<tr>
<td>3.2.1 Reduction of Concrete Strain Data</td>
<td>14</td>
</tr>
<tr>
<td>3.2.2 Strains vs. Time</td>
<td>15</td>
</tr>
<tr>
<td>3.2.3 Indoor vs. Outdoors</td>
<td>18</td>
</tr>
<tr>
<td>3.2.4 Strain Distribution</td>
<td>19</td>
</tr>
<tr>
<td>4. EVALUATION AND MODIFICATION OF PREDICTION PROCEDURE</td>
<td>21</td>
</tr>
<tr>
<td>4.1 The Prediction Method</td>
<td>21</td>
</tr>
<tr>
<td>Section</td>
<td>Page</td>
</tr>
<tr>
<td>------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>4.2 PRELOC Applied to Experimental Specimens</td>
<td>27</td>
</tr>
<tr>
<td>4.3 Comparison of Predicted and Observed Concrete Strains</td>
<td>29</td>
</tr>
<tr>
<td>4.4 Refinement of the 66-17 Procedure</td>
<td>31</td>
</tr>
<tr>
<td>4.4.1 Imperfections of the 66-17 Procedure</td>
<td>31</td>
</tr>
<tr>
<td>4.4.2 Refinement for Applied Load Effect</td>
<td>32</td>
</tr>
<tr>
<td>4.5 Result of the Refinement</td>
<td>37</td>
</tr>
<tr>
<td>4.6 Recommended Refined Procedures</td>
<td>40</td>
</tr>
<tr>
<td>5. SUMMARY AND CONCLUSIONS</td>
<td>44</td>
</tr>
<tr>
<td>6. ACKNOWLEDGMENT</td>
<td>46</td>
</tr>
<tr>
<td>7. TABLES</td>
<td>47</td>
</tr>
<tr>
<td>8. FIGURES</td>
<td>51</td>
</tr>
<tr>
<td>9. REFERENCES</td>
<td>69</td>
</tr>
</tbody>
</table>
### LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>TEST SPECIMENS</td>
<td>48</td>
</tr>
<tr>
<td>2</td>
<td>COEFFICIENTS FOR STEEL SURFACES</td>
<td>49</td>
</tr>
<tr>
<td>3</td>
<td>COEFFICIENTS FOR CONCRETE SURFACES</td>
<td>50</td>
</tr>
</tbody>
</table>
LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Plan of Experimental Bridge</td>
<td>52</td>
</tr>
<tr>
<td>2</td>
<td>Typical Cross-Section of Experimental Bridge</td>
<td>53</td>
</tr>
<tr>
<td>3</td>
<td>Framing Plane of Main Beams</td>
<td>54</td>
</tr>
<tr>
<td>4</td>
<td>Growth of Concrete Strains, Shrinkage Specimens</td>
<td>55</td>
</tr>
<tr>
<td>5</td>
<td>Growth of Concrete Strains, Short Prestressed Specimens</td>
<td>56</td>
</tr>
<tr>
<td>6</td>
<td>Growth of Concrete Strains, Beam Specimens</td>
<td>57</td>
</tr>
<tr>
<td>7</td>
<td>Vertical Distribution of Concrete Strains, Beam No. 5</td>
<td>58</td>
</tr>
<tr>
<td>8</td>
<td>Vertical Distribution of Concrete Strains, Beam No. 11</td>
<td>59</td>
</tr>
<tr>
<td>9</td>
<td>Sign Convention for Applied Loads</td>
<td>60</td>
</tr>
<tr>
<td>10</td>
<td>Predicted Long Term Concrete Strains at c.g.s., Beam Specimens</td>
<td>61</td>
</tr>
<tr>
<td>11</td>
<td>Predicted Long Term Concrete Strains at c.g.s., Short Pretensioned Specimens</td>
<td>62</td>
</tr>
<tr>
<td>12</td>
<td>Predicted vs. Observed Concrete Strains, Specimens P1 and P2</td>
<td>63</td>
</tr>
<tr>
<td>13</td>
<td>Predicted vs. Observed Concrete Strains, Specimens P3 and P4</td>
<td>64</td>
</tr>
<tr>
<td>14</td>
<td>Predicted vs. Observed Concrete Strains, Specimens 3 and 11</td>
<td>65</td>
</tr>
<tr>
<td>15</td>
<td>Predicted vs. Observed Concrete Strains, Specimens 4, 5, 9 and 10</td>
<td>66</td>
</tr>
<tr>
<td>16</td>
<td>Refinement of Prediction Procedure</td>
<td>67</td>
</tr>
<tr>
<td>17</td>
<td>Simplified Prediction Procedure</td>
<td>68</td>
</tr>
</tbody>
</table>
ABSTRACT

Long term concrete strains in pretensioned beam members in an experimental bridge were examined for an indication of the loss of pre-stress in these members under an outdoors in-service condition. The observations were used to test an existing prediction procedure for pre-stress losses.

For the highly prestressed members used in this study, prestress losses were found to be insensitive to the characteristics of the pre-stressing steel, whether stress-relieved or stabilized. The outdoor specimens showed slightly higher shrinkage and creep strains than their indoor counterparts, however, the differences were not large. The long term concrete strains predicted by the existing procedure agreed reasonably well with the field measured data. A refinement in the estimation of creep strain due to applied loads resulted in improvement agreement. Based on this refinement, prestress may be treated as remaining constant over a short period of time after application of external loads.
1. INTRODUCTION

1.1 Background

In the design of prestressed concrete members, the estimation of prestress losses is of the utmost importance, and is also one of the most difficult tasks for the design engineer. Underestimate of losses would obviously lead to premature failure of the structural member. On the other hand, overestimation could result in excessive camber, which is also objectionable.

When prestressing was first being used in concrete, the loss estimation was done by allowing a fixed amount or a fixed percentage of the initial prestress. Sources for the losses were quickly identified, but insufficient knowledge prevented accurate estimation of the various components, either separately or collectively. Research gradually led to the development of more sophisticated formulas and procedures, taking into consideration the influence of many important parameters and the interdependence of the several time dependent components. However, most of the previous research had been done using specimens fabricated and stored in laboratories with stable environmental conditions. Recently, some indication has been found that members subjected to a varying environment may behave differently from those in a stable environment. Thus, a fundamental question has been raised on whether the laboratory-based formulas can be directly applied to members intended for outdoor use.

From 1966 to 1973, an extensive research program was conducted at Lehigh University aimed at establishing a rational method for the
estimation of prestress losses in pretensioned concrete bridge members (PennDOT Research Project 66-17, Lehigh University Project FL 339). Concrete and steel specimens were tested for the material characteristics in elasticity, creep, shrinkage and relaxation. Stress-strain-time relationships for the two materials were developed by means of regression analysis of the experimental data. Combination of these relationships with linking relationships in compatibility and equilibrium then resulted in a rational procedure which enables a complete analysis of the stress and strain conditions at any time during the service life of a pretensioned member.

Inasmuch as all specimens used in that project were stored in Fritz Engineering Laboratory where the temperature and humidity conditions were reasonably stable, the question raised in the previous paragraph applied to the results. The research work reported herein, entitled "Evaluation of Prestress Loss Characteristics of In-Service Bridge Beams" (PennDOT Research Project 71-9, Lehigh University Project FL 382), was conducted primarily to provide an answer.

1.2 Scope and Objectives

The primary objective of this research project is to evaluate the usefulness of the prestress loss prediction procedure developed in the preceding project. A field study was conducted for this purpose. Several secondary studies were also included, concerning the use of newer prestressing steel elements.

The specific objectives of this project are as listed in the following:
1. To determine the prestress loss in pretensioned bridge members under an outdoor in-service condition.

2. To establish a relationship between the prestress loss behavior of laboratory specimens and in-service members.

3. To test the prediction formulas proposed by Project 66-17, and to adjust the same, if necessary.

4. To identify areas where additional research may be needed.

5. To provide an in-service comparison of the prestress loss characteristics of pretensioned bridge members containing the low-relaxation strands with those containing the stress-relieved strands.

6. To verify the transfer length of 1/2 in. diameter prestressing strands.

7. To identify the influence of strand diameter on the prestress loss of the member.

8. To identify the effect of differential shrinkage of the deck and beam concretes on the prestress loss of the beam.

1.3 Summary of Project Work

This research project was started in September 1971, in conjunction with two other research projects conducted by the Pennsylvania Transportation and Traffic Safety Center (PTTSC) of the Pennsylvania State University. The coordination of the three concurrent projects were
provided by the Pennsylvania Department of Transportation, Bureau of
Materials, Testing and Research.

The field study of this project was conducted on an experimental
bridge at the Pavement Durability Test Track facility near State College,
Pennsylvania. Concrete fiber strains in the bridge beams were measured
over a period of more than three years. Prestressed and un prestressed
control specimens were used to separate the several components of the long
term concrete strains. The measured concrete strains were analyzed and
compared with predictions based on the Project 66-17 general procedure.
Refinements to the original procedure were then made to improve the
accuracy of the prediction. This phase of the work, responding to
Objectives 1, 2, 3, 5 and 8, was performed mostly during the last three
years of the project.

The first year of this project was spent primarily on the short
term studies associated with the beginning stages of the pretensioned
members. The design of the bridge structure was done by the staff of the
PennDOT Bridge Division, and was essentially completed by October of 1971.
The fabrication of bridge members and the field construction of the
experimental bridge were performed by contractors under the concurrent
PTTSC project PennDOT Research Project 71-8, and took place in January
through July of 1972. Work by the personnel of the project being reported
(PennDOT 71-9) including the monitoring of strand tension and concrete
temperature throughout the fabrication period, and the measurement of
concrete strains at transfer time and at preselected intervals thereafter.
Analysis of these data provided information concerning the transfer length of 1/2 in. strands (Objective 6), the pretransfer loss of prestress, and a preliminary examination on the effects of strand diameter and strand type on prestress losses (Objectives 7 and 5).

Two interim reports have been issued under this research project. The first deals with a literature survey of previous studies involving prestressed concrete members stored under varied environmental conditions (I). The second interim report contains a description of the initial phase of the research as well as results through the first year.

The service load traffic test on the experimental bridge was completed in the summer of 1974, after the equivalence of more than 1.1 million applications of an 18k axle. An overload test followed, when a specially designed vehicle was used to applied successively greater loads to the bridge structure. This overload test, carried out under one of the coordinated PTTSC projects, was intended to progress until the superstructure would be judged unfit for service. It was originally hoped that flexural cracks may develop in the main beams. The load causing such cracks would be a major indication of the prestress remaining after losses. However, the capacity of the bridge structure exceeded expectation substantially. The overload testing was stopped with a maximum axle load of 120 kips, approximately four times the design standard. Under this load, the deck structure was severely cracked, but still usable. No flexural crack developed in the main beams, consequently, no direct evaluation of the remaining prestress was obtained.
1.4 Definitions

In the rapid development of prestressed concrete, a number of terms have been used rather loosely without precise and universally accepted definitions. As a result, research results in essential agreement may occasionally appear as contradicting one another. For the sake of clarity, a set of consistent definitions are adopted for this report. The author does not claim authority in pronouncing these definitions, nor does he anticipate quick acceptance by the profession. These definitions are adopted for the sole purpose of enabling a rational discussion.

**Prestress:** Prestress is defined as the stress introduced into concrete and steel prior to the application of loads. At any time after transfer, prestress is evaluated as the difference between the total stress in the material under load, and the theoretical stress caused by the applied loads. Thus, the prestress may be viewed as the stress remaining in the material if all applied loads, including the self-weight of the member, were temporarily and imaginarily removed. In this definition, the prestress in a member is not immediately changed by the application of additional loads, but the long-term effect of any sustained loading is recognized.

**Losses:** Loss of prestress is evaluated with reference to the tensioning stress in steel just before anchoring. For pretensioned fabrication, the frictional and anchorage losses are generally negligible, and the reference datum may be taken as the steel stress immediately after anchoring to the prestressing bed. The major components of prestress
losses are those due to elastic shortening, creep, shrinkage and relaxation.

1.5 Units

Unless specifically indicated, all quantities in this report are given in consistent kip-inch-day units. Strains are dimensionless, and are expressed in absolute values in calculations. In discussion, it is often convenient to express strains in microinches per inch, or microstrains, which are synonyms and equal to $10^{-6}$. 
2. EXPERIMENTAL BRIDGE

2.1 General Description

The experimental bridge used for the field study in this project is located on a pavement durability test track near State College, Pennsylvania. The bridge is a two-span two-lane structure carrying the test track over an access road. It is located on a 1% grade and a curve of 545.67 ft. radius, with a superelevation of 0.1040 ft. per ft. The superstructure consists of six pretensioned I-beams for each span, spaced at 6 ft., 10 in. c.c., and supporting a deck structure approximately 7-1/2 in. thick and 36 ft. wide between safety curbs. The span length of the beams are 60 ft. center to center of bearings.

The deck structures are varied for the two spans. For one span, the conventional cast-in-place concrete slab is used. For the other span, precast-pretensioned 3 in. planks are used, combined with a 4-1/2 in. cast-in-place concrete topping.

A detailed description of the design, fabrication, erection and testing of this experimental bridge is contained in a report by the Pennsylvania State University. A condensed version of this information, with emphasis on research Project 71-9, are contained in interim report No. 2 of this project, and again briefly recounted in Section 2.2 of this report. Figures 1, 2 and 3 show the geometry and framing plan of this bridge.
2.2 Design, Fabrication and Erection

The structural design of this experimental bridge was carried out by the staff of Pennsylvania Department of Transportation, Bridge Division, based on a service live load of the HS20-44 class and in accordance with the standard practice of that office.

For the main superstructure beams, the PennDOT standard 20/33 pretensioned concrete I-beams were selected. This section is significantly shallower than what would normally be used for the particular span and live load. The selection of this section was prompted by a desire to amplify the live load effect on the deck structure in consideration of a concurrent PTTSC project (PennDOT Research Project 71-8).

Because of the shallowness of the selected beam section, the amount of prestress is unusually high. The design required an initial prestress of 983 kips, at an eccentricity of 7.95 in. at midspan and 2.04 in. at the ends. This is supplied by thirty-four 1/2 in. prestressing strands of the 270 K grade, of which fifteen are deflected. Conventional stress-relieved strands were used in eight of the twelve beams. The other four were prestressed with the low relaxation stabilized strands. Figure 3 shows the position of these beams on the bridge.

Eight short specimens were prepared for control measurements of shrinkage and prestress strains. They all have the same cross-section as the main beams. Four are six-foot long, unprestressed, and subjected to shrinkage only. The other four are seven-foot long, and are prestressed at an eccentricity of 5.15 in. to simulate the stress condition at midspan
of the main beams under full dead load. Two of these specimens contains stress-relieved strands. The other two were prestressed with stabilized tendons. Table 1 lists all concrete specimens used in this project.

Fabrication of the beam members and short control specimens was carried out by a commercial prestresser under contract administrated by the Pennsylvania State University under the PennDOT Research Project 71-8. The handling of materials, the tensioning and detensioning of steel strands, and the placing and curing of concrete were all subjected to the standard inspection procedures of the PennDOT.

The pretensioned beams and short control specimens were fabricated during a two week period from January 26 to February 9, 1972. After transfer, all members were temporarily stored outdoors in the fabricator's yard. On May 22, 1972 (approximately 105 days after transfer), the beams were moved to the bridge site, and immediately placed on the substructures. The construction of the deck followed, and was completed 63 days later on July 24, 1972. Experimental traffic started to move over the bridge in the later part of September, and the test track was formally opened for service load on October 3, 1972.

One of each pair of the seven-foot prestressed short specimens were transported to Fritz Engineering Laboratory for long term strain measurements soon after transfer of prestress on February 15, 1972. Three six-foot shrinkage specimens were moved to the Civil Engineering Laboratory of Pennsylvania State University on March 15, 1972. The other control specimens were transported to the bridge site with the bridge beams and stored under the bridge.
2.3 **Instrumentation and Testing**

Six of the twelve main beams on the bridge, as indicated in Fig. 3, and all eight short specimens were instrumented with Whittemore gage targets for concrete strain measurements at the mid-length section. Additional targets were installed on the seven-foot prestressed specimens along the c.g.s. line. These were used in the study of transfer length.

Load cells were used on four prestressing strands (two straight and two deflected) to monitor the strand tension throughout the fabrication period from initial stretching until transfer. Concrete temperature was recorded automatically during the curing period. Whittemore gage readings of concrete strains were started just before transfer of prestress, and were continued on preselected intervals for more than three years. The last set of strain readings were taken on March 25, 1975, after the overloading test had been stopped.

2.4 **Material Properties**

The materials used for all specimens of this project were required to satisfy the quality inspection specifications of the Pennsylvania Department of Transportation. The concrete for the pretensioned beam members was specified to have a minimum compressive strength of 5000 psi at transfer time, and 5500 psi at 28 days. The prestressing strands were 1/2 in. diameter seven-wire strands of the 270 K grade. Both conventional stress-relieved strands (ASTM standard A416) and the newer low-relaxation stabilized strands were used.
The properties of the materials used in the specimens were determined by standard tests. The concrete compressive strength at transfer of prestress was 5.03 ksi on the average. At 28 days, the average strength was 7.57 ksi, significantly higher than the specified value.

The stress-relieved strands had an ultimate strength of 278 ksi, a yield strength of 248 ksi, and a secant modulus of 28600 ksi. For the stabilized strands, these properties were 278 ksi, 252 ksi, and 28,400 ksi, respectively.

The details of the testing of material properties were contained in interim report No. 23.
3. ANALYSIS OF EXPERIMENTAL RESULTS

3.1 Short-Term Observations

One of these short term studies dealt with the change of strand tension before transfer. Load cell measurements of tension in four of the thirty-four strands showed that the strand force was slightly lower than expected at the beginning, and that the total loss prior to transfer was compatible to the anticipated relaxation loss for this period of time. During the curing period, when the concrete temperature was raised from approximately 50°F to approximately 130°F, the strand tension showed a significant drop. However, at the end of curing, the tension increased as temperature decreased, and the net effect of this excursion into high temperature was negligible.

The second short term study dealt with the transfer length of the 1/2 in. pretensioned strands. This was accomplished by measuring the concrete strain along the c.g.s. line of the short prestressed specimens. Analysis of these data showed that after transfer, the total pre-stress force was developed from zero at the exposed ends to its maximum value in the middle portion over a length of approximately 30 in. The maximum prestress level was in agreement with the conventional elastic analysis, based on the load cell measurement of tendon force just before transfer. The indicated transfer length, of approximately 30 in., was also in agreement with the current recommendations of the AASHTO\(^1\) and ACI\(^2\) specifications.

-13-
Both in the transfer length study, and in the analysis of time variations of concrete strains in the prestressed specimens, no significant difference was found between the member containing the stress-relieved strands and those containing stabilized strands.

In addition to the above, observations at the transfer time and during the first year also confirmed the linear distribution of concrete strains across the depth of pretensioned members, which is a basic assumption used in the development of the general predicting procedure in the research project 66-17.

3.2 Long Term Observations

3.2.1 Reduction of Concrete Strain Data

For long range purposes, the Whittemore gage readings immediately after transfer were used as the basis for reference, and the total concrete strains after transfer were calculated and analyzed. The Whittemore strain gage used for the measurements has a gage length of 10 in., and a dial gage with 0.0001 in. divisions. Thus, the precision for each strain reading is $10^{-5}$ in. per in., or 10 microstrains. When several strain readings were averaged, the degree of precision was improved somewhat.

Four studies were made on these long term strain data. They are:

1. Variation of measured concrete strain with time, aimed at the comparison between specimens containing the two types of prestressing elements.
2. Comparison of observed concrete strains in the indoor and outdoor paired specimens.

3. Variation of concrete strains across the depth of the member.

4. Comparison of the experimental concrete strains with the values predicted by the project 66-17 procedure.

3.2.2 Strains vs. Time

Figures 4, 5 and 6 illustrate the variation of measured concrete strains in the three groups of specimens over the three-year period. A general trend of strain growth in decreasing rate with time is evident in all three figures. When empirical data were plotted against time on a natural (straight) scale, the gradually decreasing rate of strain growth and the increasing time interval between measurements made it extremely difficult to determine whether or not the strain growth was continuing, particularly in view of the unavoidable scattering of data. However, when logarithmic scale is used for time, as in the figures contained in this report, the continued growth of strain over the entire three year period is unmistakable. Moreover, the nearly linear relationship between strain and the logarithm of time can also be visualized. These observations are in agreement with those presented in several previous reports$^6,7,8$.

Figure 4 shows the average strain in each of the four shrinkage specimens. Keeping in mind the age differential between the two fabrication runs, it is seen that specimens S1, S2 and S3 behaved very similarly through the whole period, while specimen S4, which remained outdoors after
the other three had been moved inside (at approximately 80 days) showed somewhat more fluctuation from the general growing trend. Quantitatively, the average strain in specimens S1, S2 and S3 was approximately 200 microstrains at three days, growing to 300 microstrains at 80 days, 350 microstrains at 250 days, and 450 microstrains at 1200 days. On the other hand, although specimen S4 also exhibited a strain of 300 microstrains at 80 days, after that time the strain fluctuated between 250 and 350 microstrains with a very low trend of long term growth.

Figures 5 and 6 show the long-term concrete strains at a gage level very close to the c.g.s. line of the short pretensioned specimens and the beam specimens, respectively. In both cases, a nearly linear relationship between strain and the logarithm of time can be observed in agreement with prediction based on the previous research. In the short prestressed specimens, the long term strain was approximately 400 microstrains at ten days, between 800 and 1100 microstrains at one year, and between 850 and 1200 microstrains at three years. Compared with an initial elastic strain of approximately 850 microstrains, these indicate a growth of about 100% over the three year period, nearly 90% of which occurred during the first year. Almost identical observations can be made of the prototype beam specimens.

Figures 5 and 6 are also used to show the effect of the characteristic of steel, viz.: stress-relieved strands versus stabilized strands. The two indoor short prestress specimens, P2 and P3, behaved almost identically over the initial 150 days, after which P3 containing
stabilized strand showed more strain growth. The long-term strain in both specimens were approximately 400 microstrains at 7 days and 830 microstrains at 150 days. At the end of one year, the strains were approximately 1080 and 970, respectively. At the end of the third year, the long-term strains had each gained approximately 100 microstrains, to 1180 and 1060 microstrains, respectively. The two outdoors short prestressed specimens, P1 and P4, showed different behaviors from the very beginning. At ages of 7 days, 150 days, 1 year and 3 years, the strains in these two specimens were approximately 500 vs. 300, 730 vs. 620, 820 vs. 730, and 950 vs. 800 microstrains, respectively. It should be pointed out that in this pair, the specimen containing stress-relieved strands, exhibited higher concrete strains, which is contrary to the comparison of the indoors specimens.

Figure 6 shows virtually no difference between the behavior of specimens containing the two types of prestressing strands. Specimens 3 and 11, containing stress-relieved strands, registered slightly lower concrete strains than the other specimens. However, the difference is extremely small, and cannot be considered significant in view of the unavoidable scattering of data.

As noted previously⁹, this lack of consistent and pronounced differences between the two types of strands was unexpected but not unreasonable. In Section 2.2, it has be pointed out that the beam section is very heavily prestressed. (The calculated initial concrete prestress at c.g.s. is nearly 3.5 ksi.) In the preceding project (PennDOT 66-17), it has been established that for very heavily prestressed members, the
prestress losses are controlled primarily by the concrete characteristics, and relaxation accounts for only a small portion\(^6\). It is therefore understandable that all specimens behaved nearly the same regardless of the character of the steel elements. It is unfortunate that the beam design selected to facilitate the structural study (PennDOT 71-8) in effect rendered it insensitive to the strand characteristics on prestress losses. Even so, the observation should be made that in general, specimens containing stabilized strands showed more long term concrete strain, indicating more remaining prestress, than their counterpart containing stress-relieved strands. Therefore, it is reasonable to postulate that the use of stabilized strands probably has a beneficial effect with regard to prestress losses.

3.2.3 Indoors vs. Outdoors

The effect of the changing environment on concrete strains are observed from Figs. 4 and 5 by comparing the behaviors of specimen S3 with S4, P2 with P1, and P3 with P4. In each pair, the first specimen was kept indoors under a rather constant condition, and the second member was influenced by the outdoor varying environment.

In all three cases, the indoor specimen registered more concrete strain than the outdoor one. At the end of three years, S3 has shown a total shrinkage of 380 microstrains as compared to 340 microstrains in S4. The short prestressed members showed more clear differences. At the level of c.g.s. line, P2 registered a total long term strain of 1060 microstrains in three years, in contrast to 950 units in specimen P1.
The comparisons between P3 and P4 are more pronounced, when the long term strain values after three years were 1180 and 800 microstrains, respectively.

It can be concluded that outdoor structural members will suffer less shrinkage strain than specimens stored under constant (and rather dry) conditions in a laboratory. However, as will be shown in Chapter 4 of this report, the significance of the environmental difference is not certain.

3.2.4 Strain Distribution

The vertical distribution of measured concrete strains over the depth of the prestressed concrete specimens was studied by plottings exemplified by Figs. 7 and 8. The nearly linear distribution at all times is clearly shown. In addition, the gradual increase of curvature with time and the abrupt change of curvature caused by the application of superimposed load (bridge deck slab) are also evident. All of these observations are in agreement with the common basic assumption for prestressed concrete analysis, that is, the section may be treated as if made of a linear elastic material.

It would be pointed out that these figures show the long-term concrete strains, occurring after the transfer of prestress. The strains developed at the time of transfer, primarily caused by elastic shortening of concrete, have been excluded. An examination of those initial stresses has been included in the previous interim report. Their agreement with the linear elastic theory has been demonstrated.
The verification of the linear strain distribution in the specimens is very significant, as it enables an estimation of the steel strain in the concrete specimens at any time. The concrete strains were measured at four different levels in each specimen. However, the geometry of the cross-section and the eccentricity of prestress made it inconvenient to measure strains directly along the c.g.s. line (except for the short prestressed specimens). Consequently, the strain in the prestressing steel was not measured directly, but can only be estimated by means of the linear law of strain distribution. In Chapter 4, the steel strains estimated in this manner were compared with predicted values based on the method developed in Project 66-17 in an effort to evaluate the usefulness of that method.
4. EVALUATION AND MODIFICATION OF PREDICTION PROCEDURE

4.1 The Prediction Method

The prediction method, previously developed in research project PennDOT 66-17, is based on the basic proposition that the long-term characteristics of the steel and concrete materials can be approximately represented by their respective stress-strain-time relationship, as follows:

\[
\frac{f_s}{f_{pu}} = \frac{A_1}{A_2} + \frac{A_3}{A_2} \varepsilon_s + \frac{A_3}{A_2} \varepsilon_s^2 - \frac{B_1}{B_2} \log (t_s + 1) \varepsilon_s - \frac{B_3}{B_4} \log (t_s + 1) \varepsilon_s^2
\]

(4-1)

and

\[
\varepsilon_c = C \varepsilon_s + \left[ \frac{D_1}{D_1} + \frac{D_2}{D_2} \log (t_s + 1) \right] + \left[ \frac{E_1}{E_3} + \frac{E_2}{E_4} \log (t_c + 1) \right] + f_c \left[ \frac{E_3}{E_4} + \frac{E_4}{E_4} \log (t_c + 1) \right]
\]

(4-2)

where

- \( f_s \) = Stress in prestressing steel, tension positive
- \( f_{pu} \) = Specified ultimate tensile strength of prestressing steel
- \( \varepsilon_s \) = Strain in prestressing steel, tension positive, in \( 10^{-2} \)
- \( t_s \) = Steel time, starting from initial tensioning
- \( f_c \) = Compressive stress in concrete, compression positive
- \( \varepsilon_c \) = Strain in concrete, compression positive, in \( 10^{-2} \)
- \( t_c \) = Concrete time, starting from the time of prestress transfer, which is taken to be the same as the end of curing (hence, both shrinkage and creep start at \( t_c = 0 \))
The A, B, C, D, and E's in the above equations are regression coefficients evaluated empirically in the previous research project, and are listed in Tables 2 and 3.

For pretensioned concrete members, the material characteristic equations (4-1) and (4-2) are connected by three sets of linking relationships, namely

**Time Compatibility**

\[
T_s - T_c = k_1
\]  

(4-3)

**Strain Compatibility**

\[
\varepsilon_s + \varepsilon_c = k_2
\]  

(4-4)

**Static Equilibrium**

\[
\int_c f_dA_c - \int_s \varepsilon_s a_{ps} = P
\]  

(4-5)

\[
\int_c f_dA_c - \int_s \varepsilon_s a_{ps} = -M
\]  

(4-6)

In the above equations:

\[
k_1 = \text{Time interval from tensioning of steel to transfer of prestress}
\]

\[
k_2 = \text{Initial tensioning strain in steel, in } 10^{-2}
\]

\[
A_c = \text{Area of net concrete cross-section}
\]

\[
x = \text{Distance to elementary area, from the centroidal horizontal axis of the gross cross-section}
\]

\[
a_{ps} = \text{Area of individual prestressing element}
\]
P = Total thrust on section, caused by external loads
M = Bending moment on section caused by external loads

The positive directions of x, P and M, as well as the elementary areas dA_c and a_p, are shown in Fig. 9.

The linear distribution of strains, discussed in Section 3.2.4, when combined with the linear stress-strain relationship for concrete, Eq. 4-2, implies that the concrete stress distribution is also linear. Therefore,

\[ f_c = g_1 + g_2 x \]  \hspace{1cm} (4-7)

when \( g_1 \) and \( g_2 \) are stress parameters. In the general procedure, the equations (4-1) through (4-7) are combined into two simultaneous quadratic equations in \( g_1 \) and \( g_2 \), of which the coefficients are functions of the material constants, the linkage parameters \( k_1, k_2 \), P and M, and a time parameter, say \( t_c \). For any specified time, the coefficients can be evaluated, the solution of these simultaneous equations for \( g_1 \) and \( g_2 \) then enables the estimation of \( f_c \) in any fiber, and subsequently the steel stress in any element.

For practical purposes, the dispersed steel area may be treated as concentrated at the c.g.s., and the equations can be simplified considerably. In the final report of the previous project\(^6\), it has been shown that the controlling equation in this case is

\[ (R_1 - \beta f'_c) + (R_2 - \beta + 1) f_{cs} + R f'^2_{cs} = 0 \]  \hspace{1cm} (4-8)
In the above equation,

\[ f_{cs} = \text{Concrete fiber stress at the level of c.g.s.} = g_1 + g_2 e, \text{ compression positive} \]

\[ f'_{cl} = \text{Nominal concrete fiber stress at c.g.s. caused by applied loads, tension positive} = -\frac{P}{A} + \frac{M e}{I g} \]

\[ R_1, R_2, R_3 = \text{Functions of the material characteristic constants, } k_1, k_2 \text{ and } t_c \text{ (fully defined in Ref. 8)} \]

\[ A_g, I_g, e_g = \text{Area, moment of inertia, and eccentricity, respectively, of the gross cross-section} \]

\[ \beta = \left( \frac{1}{A \frac{1}{A} + \frac{e^2}{I g}} \right) \]

and \[ A_{ps} = \text{Total area of prestressing steel} \]

Combination of equations (4-8), with (4-1), (4-2) and (4-4) results in a simpler equation for the evaluation of steel stress

\[ f_s = (\beta - 1) f_{cs} + \beta f'_{cl} \]

It should be emphasized that the \( f_s \) in the foregoing is the total steel stress, including the effects of all applied loads. As defined in Section 1.4, the prestress is evaluated by removing these load effects from \( f_s \). In accordance with the linear-elastic theory, the stresses caused by applied loads are:
where \( f_{cl} \) = Actual concrete stress at c.g.s. caused by applied loads, compression positive

\( f_{s\lambda} \) = Steel stress caused by applied loads, tension positive

\( n \) = Modular ratio of steel to concrete

Therefore, the steel prestress at a given time is

\[
f_p = f_s - f_{s\lambda} = (\beta - 1) f_{cs} + \left( \beta - \frac{n\beta}{\beta + n - 1} \right) f'_{cl}
\]

\[
= (\beta - 1) [f_{cs} - f_{cl}]
\]

The loss of prestress is, by the definition given in Section 1.4

\[
\Delta f_p = f_{si} - f_p
\]

where \( f_{si} \) is the initial steel stress at times \( t_s = 0 \).

In summary, the basic procedure developed in Project 66-17 for the estimation of prestress loss in pretensioned members involves the following steps:

1. Material characteristic constants, geometry, loading, and fabrication parameters are given.
2. For specified time \( t_c \), evaluate \( R_1 \), \( R_2 \) and \( R_3 \)
3. Solve equation (4-8) for \( f_{cs} \)
4. Evaluate steel stress $f_s$ by equation (4-9)

5. Concrete and steel strains, if needed, are calculated by equations (4-2), and (4-4), respectively.

6. Steel prestress and loss of prestress are calculated by equations (4-12) and (4-13).

A computer program PRELOC was developed to carry out these calculations. For the convenience of discussion, six key stages in the life of a pretensioned concrete beam member are identified, and the computer program PRELOC was set up to determine the stress and strain conditions at each of these six stages plus additional intermediate times. The six stages, indicated by subscripts to the stress, strain and time parameters, are the following:

1. Initial tensioning stage, immediately after anchorage to pretensioning bed: $t_s = 0$, $f_s = f_{si}$, $e_s = k_s$

2. Immediately before transfer: $t_s = k_1$, $e_s = k_2$, $f_s < f_{si}$

3. Immediately after transfer, including the effect of self-weight:
   $t_s = k_1$, $t_c = 0$, $P = 0$, $M =$ self weight moment

4. Immediately before application of additional load: $t_c > 0$, $P = 0$, $M =$ self weight moment

5. Immediately after application of additional load: $t_s = t_4$, $P$ and $M$ include the effect of added loads

6. End of service life, taken as 100 years; $t_c = 36500$, $P$ and $M$ same as at stage 5
4.2 PRELOC Applied to Experimental Specimens

The general procedure described in the preceding section was applied to the specimens used in this field study, in order that its usefulness can be evaluated. As the primary purpose of these research efforts was to estimate prestress losses, it would be ideal if predicted and observed values of the losses could be compared. However, there is no practical technique for the direct measurement of prestress losses, or the prestress after fabrication. It is also felt that for the comparison to be meaningful, the observed values should be used as directly as possible from actual measurement. For this reason, it was decided to use the long term concrete strain at c.g.s. as the comparing parameter. The "experimental" values of this quantity are obtained from the corresponding values at the strain gage location by means of linear interpolation. The prediction values were calculated by the PRELOC program described in the previous section.

Four sets of prediction calculations were made by means of the PRELOC program. These included both the prototype bridge beams and the short prestressed specimens, and covered both types of steel strands. The concrete material was identified as corresponding to the lower bound of prestress losses. The steel characteristics are that of the stress-relieved strands or the stabilized strands, as the case may be. The other input data, based on design and fabrication information, are as the following:
Geometry: 
\[ A_g = 417 \text{ sq. in.} \]
\[ I_g = 44757 \text{ in.}^4 \]
\[ A_{ps} = 5.20 \text{ sq. in.} \]
\[ e_g = 7.95 \text{ in.} \quad \text{For the prototype beams} \]
\[ 5.15 \text{ in.} \quad \text{For short prestressed specimens} \]

Fabrication: 
\[ f_{sl} = 186 \text{ ksi} \]
\[ k_2 = 0.635 \times 10^{-2} \]
\[ k_1 = 2.3 \text{ days} \]

Loading: 
For prototype beams:
- Initial load: \[ M = 2340 \text{ k-in. (self weight)} \]
- Subsequent load:
  - For Beams 4, 5, 9 and 10: \[ t_{c5} = 137 \]
    \[ M = 6448 \text{ K-in.} \]
    \[ P = -28.65 \text{ k} \]
  - For Beams 3 and 11: \[ t_{c5} = 142 \]
    \[ M = 6408 \text{ k-in.} \]
    \[ P = -28.33 \text{ k} \]

Note: The wearing surface on the bridge is supported by the composite section, resulting in a net tensile load in the precast beam section.

For short prestressed beams, 
\[ P = 0 \]
\[ M = 0 \]
Results of the computer progress calculations are shown in Figs. 10 and 11. It is seen that the strand type (stress-relieved or stabilized) has rather insignificant effect on the predicted behavior of the specimens. As previously indicated, this is because of the very high prestress used in the specimens ($\beta = 50.5$ for the prototype specimens and 64.3 for the short specimens). A discontinuity is shown in Fig. 10, at time $t_4 = t_5$, corresponding to the application of superimposed dead load, which was treated as occurring instantaneously in the prediction calculation.

4.3 Comparison of Predicted and Observed Concrete Strains

The predicted and observed long-term concrete strains in the prestressed specimens are compared in Figs. 12 through 15. In each figure, the predicted values are shown by the solid line, and the isolated data points represent the observed values at various times. For the short specimens, these are averages of the measured values taken directly at the c.g.s. level. For the prototype beams, concrete strains were not directly measured at the c.g.s. level, and the plotted values were calculated by imposing the linear strain distribution condition.

Figures 12 and 13 show the comparison for the short prestressed specimens. The comparisons are reasonably good for all specimens. Actually, the comparison appears slightly better for the outdoor specimens P1 and P4, rather than the indoor specimens, P2 and P3. However, the differences are not large. In view of the scatter of data, it is felt that
not too much significance should be attached to this indoor-outdoor separation. Instead, the prediction method is seen to be applicable to outdoor specimens with as much accuracy as to the indoor specimens.

Figures 14 and 15 show the comparison of predicted and observed concrete long-term strains in the prototype specimens. The comparison is again seen to be reasonably good for all specimens, particularly before the application of superimposed load (the casting of deck slab). While the 66-17 procedure predicted an abrupt change of concrete strain at this time, the observed data did not show such discontinuity. This discrepancy can be attributed, at least partially, to two factors. The first is the scatter of the observed data. Inasmuch as that the observed data points do not lie on a smooth curve, a slight discontinuity is not easily detectable. Secondly, the 66-17 prediction procedure does contain a short-term imperfection at this point. A detailed discussion of this imperfection, and the refining of the procedure to remove the same, will be presented in subsequent sections of this report. Following the application of load, the prediction curve tends to be below the observed data. The underprediction of long-term concrete strain implies lower predicted concrete stresses causing creep, which in turn reflect lower long-term prestressing force or higher predicted prestress losses. Quantitatively, the strain differentials are in the order of 125 microstrains, corresponding to a steel stress differential of approximately 4 ksi or 3% of the predicted final steel stress.
4.4 Refinement of the 66-17 Procedure

4.4.1 Imperfections of the 66-17 Procedure

The comparison between the calculated and observed concrete strains, described in Section 4.3, showed a discrepancy which, though not excessive, should preferably be reduced. An examination of the previously developed procedure revealed two specific imperfections.

The first imperfection of the previous procedure concerns the shrinkage strain estimation. All specimens of Project 66-17 were stored in Fritz Engineering Laboratory, under a relatively dry and stable environment. The more humid and varied outdoor environment is expected to cause a somewhat lower shrinkage strain in concrete, and corresponding lower pre-stress losses. This was originally the primary objective of this research project. However, as discussed in the previous section, and shown in Figs. 12 and 13, the environmental effect appears not to be significant.

An effort was made to adjust the shrinkage strain coefficients ($D_1$ and $D_2$ in Eq. 4-2) in accordance with the humidity condition. Based on shrinkage information available in literature, a multiplier was applied to the shrinkage coefficients $D_1$ and $D_2$. As the coefficients $D_1$ and $D_2$ were based on shrinkage data corresponding to an average relative humidity of approximately 50%, the multiplier is larger than 1.0 for drier environment and less than 1.0 for wetter environments. Although the approach was reasonable, the application of such a multiplier resulted in no improvement of the prediction. In fact, in Section 4.3, it has been pointed out that the prediction agreed slightly better to the outdoor specimens than to the...
indoor specimens. It was concluded that there is no practical merit in adjusting the shrinkage strains in this manner, and the coefficients $D_1$ and $D_2$ were left unchanged for the remainder of this project.

The second imperfection of the previous procedure concerns the short term effect of the load applied after transfer. In the original procedure, for the analysis at a given time, all loads present at that time were treated as if continuously acting from the time of prestress transfer ($t_c = 0$). This approach clearly overestimates the creep effect of all applied loads except the member's own weight. As a matter of fact, each time a load is added to the member, this approach will reflect a sudden decrease in concrete strain and a corresponding increase in steel prestress. These discontinuities in prestress are obviously inconsistent with the basic definition of prestress given in Section 1.4. This imperfection in the 66-17 prediction procedure can be substantially corrected by revising the concrete strain equation (4-2). An outline of the involved refinement is given in the following Section 4.4.2, and a detailed description is contained in Ref. 5.

4.4.2 Refinement for Applied Load Effect

In Eq. 4-2, the creep component of concrete strain is estimated on the basis of total current concrete stress $f_c$, and the total time from transfer $t_c$. In reality, a part of the concrete stress is caused by loads applied much later. The creep effect of this part obviously does not start before the application of these loads. Therefore, a correction is needed in Eq. 4-2 to remove the non-existent creep strain attributed to the
applied load stresses prior to their presence. In particular, consider an increment of concrete stress, $\Delta f_c$, applied at time $t_4 = t_5$, the total creep strain at a time $t_c > t_5$ can be estimated as

$$\varepsilon_{cr} = E_f + E_{f_c} \log (t_c + 1) + [E_3 + E_4 \log (t_c + 1)] (f_c - \Delta f_c)$$

$$+ \left[ E_3 + E_4 \log (t_c - t_{c5} + 1) \right] \Delta f_c$$

Here $f_c = $ Total concrete stress, including the effect of prestress, self weight of member, as well as subsequently applied load

$\Delta f_c = $ Concrete stress caused by loads applied at time $t_5$

Both $f_c$ and $f_{cd}$ are positive when in compression. When this expression for creep strain is combined with those for elastic and shrinkage strains, it is more convenient to rearrange the last two terms, resulting in the revised concrete surface equation as the following:

$$\varepsilon_c = E_f f_c + [D_1 + D_2 \log (t_c + 1)]$$

$$+ \left[ E_1 + E_2 \log (t_c + 1) \right] + \left[ E_3 + E_4 \log (t_c + 1) \right] f_c$$

$$- E_4 \left[ \log (t_c + 1) - \log (t_c + 1 - t_{c5}) \right] \Delta f_c$$

Equation 4-14 obviously applies only when $t_c \geq t_{c5}$. Otherwise, the last term should be dropped, and the equation reverts to (4-2). It is interesting to note that the corrective term in (4-14) has its largest value when $t_c = t_{c5}$, and becomes smaller as $t_c$ increases, approaching zero as a
limit. This is a very natural trend, as the time differential becomes less and less important as time progresses.

The revised prediction procedure involves using Eq. 4-2 for concrete stress-strain-time relationship for \( t_c \leq t_c^4 \) and using Eq. 4-14 for \( t_c \geq t_c^5 \). The continuity of prestress at time \( t_c^4 = t_c^5 \) is assured by evaluating \( f_{cd} \) in accordance with this condition. The concrete strains at times \( t_c^4 \) and \( t_c^5 \) are evaluated by Eqs. 4-2 and 4-14, respectively.

\[
\varepsilon_{c_4} = C_{1} f_{c_4} + \left[ D_1 + D_2 \log (t_c^4 + 1) \right] + E_1 + E_2 \log (t_c^4 + 1) + \left[ E_3 + E_4 \log (t_c^4 + 1) \right] f_{c_4}
\]

\[
\varepsilon_{c_5} = C_{1} f_{c_5} + \left[ D_1 + D_2 \log (t_c^5 + 1) \right] + E_1 + E_2 \log (t_c^5 + 1) + \left[ E_3 + E_4 \log (t_c^5 + 1) \right] f_{c_5}
\]

\[
- E_4 \left[ \log (t_c^5 + 1) - \log (t_c^5 + 1 - t) \right] \Delta f_c
\]

Subtract \( \varepsilon_{c_4} \) from \( \varepsilon_{c_5} \), and recognizing that

\[
t_c^5 = t_c^4
\]

and \( f_{c_5} = f_{c_4} + \Delta f_c \)

the instantaneous change of concrete strain is related to stress change in the following manner:

\[
\varepsilon_{c_5} - \varepsilon_{c_4} = (C_1 + E_3) (f_{c_5} - f_{c_4})
\]

\[
= (C_1 + E_3) \Delta f_c \tag{4-15}
\]
The strain compatibility condition, Eq. 4-4, requires that the change of strain in prestressing steel is equal to the strain change in concrete at the same level, but with the opposite sign.

\[ \varepsilon_{s5} - \varepsilon_{s4} = -(C_1 + E_3) f_{cd} \]  \hspace{1cm} (4-16)

where \( f_{cd} \) = change of concrete fiber stress due to applied load at the level of c.g.s.

The instantaneous change of steel stress is calculated by the steel stress-strain-time relationship, Eq. 4-1

\[ \Delta f_s = P_2 (\varepsilon_{s5} - \varepsilon_{s4}) + P_3 \varepsilon_{s5}^2 - \varepsilon_{s4}^2 \]

\[ = (P_2 + 2P_3 \varepsilon_{s4}) (\varepsilon_{s5} - \varepsilon_{s4}) + P_3 (\varepsilon_{s5} - \varepsilon_{s4})^2 \]  \hspace{1cm} (4-17)

where \( P_2 = A_2 - B_2 \log (t_{c4} + k_1 + 1) \)

\[ P_3 = A_3 - B_3 - B_4 \log (t_{c4} + k_1 + 1) \]

The relationship between the increment stresses, \( \Delta f_c \) and \( \Delta f_s \), and the increment loads, \( \Delta P \) and \( \Delta M \), is established by imposing the equilibrium conditions. These increment loads and stresses must satisfy static equilibrium conditions similar to Eqs. 4-5 and 4-6. In addition, since concrete stress distribution is linear before and after the load application, the linear relationship (4-7) also holds for the increment concrete stress \( \Delta f_c \). Considering the simplified but practical case where all steel is treated as concentrated at the c.g.s. level, the equilibrium equations are reduced to the following forms:
Multiply Eq. 4-18a by \( I_g \), Eq. 4-18b by \( A e_g \), add the two equations together, and substituting Eq. 4-19

\[
A I_g f_{cd} - A_{ps} (\Delta f_s + f_{cd}) \left( I_g + A e_g^2 \right) = \Delta P - \Delta M
\]

Divide through by \( A I_g \), and introducing parameters \( \beta \) and \( \Delta f'_{c\ell} \), as previously defined (Eq. 4-8):

\[
f_{cd} - \frac{1}{\beta} (\Delta f_s + f_{cd}) = -\Delta f'_{c\ell}
\]

Combining this equation with Eqs. 4-16 and 4-17, a quadratic equation in \( f_{cd} \)

\[
-\beta \Delta f'_{c\ell} + (P_2 + 2P_3 e S_4) (C_1 + E_3) - \beta + 1 \right) f_{cd} + P_3 (C_1 + E_3)^2 f_{cd}^2 = 0
\]

Equation 4-20 is similar to Eq. 4-8, and defines the appropriate value of \( f_{cd} \) to be used in Eq. 4-14 in terms of the nominal stress increment \( \Delta f'_{c\ell} \). It is emphasized here that \( \Delta f'_{c\ell} \), like \( f'_{c\ell} \) defined previously, is positive when in tension, and therefore has a reverse sign convention as compared to \( f_{cd} \).

The revised procedure may be summarized in the following steps:

1. For concrete time \( t_c < t_{c4} \). There is no change to the previous 66-17 procedure.
2. At time $t_5 = t_s$, the increment loads $\Delta P$ and $\Delta M$ cause an increment nominal concrete stress $\Delta f'_{cl}$, and a corresponding time concrete stress $f_{cd}$ at the c.g.s. level. The value of $f_{cd}$ is solved from Eq. 4-20.

3. For $t_c < t_s$, use Eq. 4-14 in place of Eq. 4-2, but otherwise follow the same procedure as before.

The computer program PRELOC, previously developed for the Project 66-17 procedure, has been expanded to include the above described modification. The new program is called PRELOI, which includes a subroutine FSAD for the evaluation of $f_{cd}$ at the loading time $t_5$. Comparison of the results from this modified program with those from PRELOC and field observations are given in the following section.

4.5 Result of the Refinement

The effect of the refinement for applied load described in Section 4.4.2 is clearly depicted in Fig. 16, which shows the prestress loss $(f_{si} - f_p)$ in the Test Track Bridge Beams predicted by the original as well as the new procedures. As expected, the two predictions approach each other asymptotically as time increases. For the most part, the two predictions differ by less than 0.5 ksi, or approximately 1% of the predicted losses. For all practical purposes, difference in prestress losses of such magnitude can be safely ignored, and indeed should be in view of the tolerated variations of material properties and fabrication inaccuracies. Before the application of load at time $t_s$, the two procedures give identical results. While the original procedure yields a discontinuity
at \( t_s \), the prediction by the new procedure is continuous. It is interesting to note that the new procedure also predicted a temporary decrease in prestress loss, but the trend is reversed after a short time interval, and the extend of this decrease is considerably less than the discontinuity predicted by the original procedure. In the example shown, the original procedure predicted a sudden change in prestress of 3.3 ksi (approximately 2% of prestress). By the new procedure, the decrease grew gradually and reached a maximum of 1.2 ksi in about 30 days, and returned to the pre-load level after approximately 150 days. This excursion of prestress loss is explained by the delayed response of concrete under the stress change caused by applied loads. Immediately following loading, the creep behavior of concrete is dominated by the newly applied stress, the effect of the previous existing stress is overshadowed. As time progresses, however, the effect of new stress decreases more rapidly than that of the original stresses, since the growth of creep strain with time is not linear but logarithmic. Therefore, after a period of time, the trend of prestress change returns to the normal gradual decrease with time, and the difference between the two prediction procedures diminishes indefinitely.

The effect of the refinement made in procedure PRELO1 is demonstrated in Figs. 14 and 15 where predicted long term concrete strains \((\epsilon_c - \epsilon_{c_3})\) at the c.g.s. level are compared with field observations and predictions by the old procedure PRELOC. In these figures, predictions by the refined procedure are represented by the dashed lines. It is clearly seen that the agreement between predicted and observed values is significantly improved by the refinement.
A study was made on the duration and extent of the prestress excursion immediately after the application of loads. It is clear that both of these quantities are dependent upon the magnitude of the newly applied concrete stress \( f_{cd} \) and the age of concrete when the stress is applied \( t_{cs} \). If either of these two parameters is zero, there will be no excursion, and prestress loss will grow monotonously throughout the life of the member. For larger \( f_{cd} \) or \( t_{cs} \) or both, the prestress excursion will be more severe and also extend over a longer period of time. A parametric study was made, with \( f_{cd} \) varying from 0.20 to 1.0 ksi and \( t_{cs} \) varying from 90 to 210 days. The largest increase of prestress during the excursion period was found to be less than 3 ksi, approximately 2% of the prestress at the time of loading. The length of the prestress excursion period (\( \Delta t \) in Fig. 16) was found to vary from 25 days for small loads applied at early stage to nearly 800 days for the opposite combination. In view of the uncertainties involved in prestress estimation, it is felt that the small increase of prestress could be ignored, and the prestress may be treated as remaining constant during the excursion period. When the full computerized general procedure (PRELOI) is used, such a simplification is of little value, since the saving in computer time is very small. On the other hand, if calculations are to be done manually treating prestress as remaining constant during the excursion period will significantly reduce the amount of work and is fully justifiable.

An attempt was made to develop an empirical equation for the duration of prestress excursion period, \( \Delta t \), in terms of \( f_{cd} \) and \( t_{cs} \). It was found that \( \Delta t \) is also influenced by the geometrical design parameter \( \beta \)
of the member section. The empirical expression obtained from an extensive analysis of the parametric study data was too complicated for practical use. However, an approximate estimate of $\Delta t$ can be made quite easily based on the linear relationship of prestress loss and the logarithmic of time. This will be further explained in Section 4.6.

4.6 Recommended Refined Procedures

Following the discussions presented in Section 4.5, it is evident that the refinement of the concrete characteristic relationship, Eq. 4-14, results in a significant improvement of the prediction of prestress losses in pretensioned concrete bridge members. It is therefore recommended that the refined procedure, represented by computer program PRELOI, fully described in Section 4.4, be used by designers of these members. The simplification described in the last part of Section 4.5 is not recommended for incorporation into the computer program. If manual calculations are intended, the approximation of treating prestress as remaining constant during a period of time after loading is recommended. The refined simplified method, suitable for manual usage, is similar to that recommended by the previous project. It includes the following steps (Fig. 17):

Input data required: Concrete material characteristics

- Initial tensioning stress $f_{si}$
- Transfer time $k_1$
- Geometric design parameter $\beta$
Nominal concrete stress at c.g.s. caused by full long term load $f'_{cl}$

Nominal concrete stress at c.g.s. caused by loads, applied later than transfer of prestress $\Delta f'_{cl}$

Age of concrete when $\Delta f'_{cl}$ is applied $t_{cs}$

Step 1: Initial prestress loss, at transfer time:

$$IL = REL + EL$$

The two parts of initial loss IL are:

$$REL = \text{Pre-transfer relaxation loss, dependent upon}$$
$$f_{sl} \text{ and } k_1 \text{, and calculated from the steel stress-strain-time relationship, Eq. 4-1}$$

$$EL = \text{Elastic loss of prestress}$$

$$EL = \frac{n_1}{n_1 + \beta - 1} (f_{sl} - REL)$$

where $n_1 = \text{Initial modulus ratio}$

Step 2: Final prestress loss, taken at the end of 100 years

$$TL = SRL + ECR - LD$$

The through components of the final loss TL are:

$$SRL = \text{Component independent of concrete stress, dependent upon concrete characteristics and } f_{sl}$$
ECR = component directly dependent upon concrete stress

= 2.2 EL

LD = Effect of applied load, including weight of members

= \( (\gamma - 1) \frac{n_i \beta}{\beta + n_i - 1} f'_c \)

where \( \gamma = 2.9 \) for lower bound losses

3.3 for upper bound losses

Step 3: Auxiliary final prestress loss: If the load creating \( \Delta f'_c \) were eliminated, the member would be under a lighter load over its entire life, and the final loss would be higher

\[ T_{L_D} = T_L + \Delta LD \]

\[ \Delta LD = (\gamma - 1) \frac{n_i \beta}{n_i + \beta - 1} (\Delta f'_c) \]

Step 4: Loss of prestress at intermediate time \( t_c \)

(a) Before the application of load, \( t_c < t_{c5} \)

\[ PL = IL + 0.22 (T_{L_D} - IL) \log t_c \]

(b) Duration of prestress excursion period, \( \Delta t: \)

\[ \frac{\log (t_{c5} + \Delta t)}{\log t_{c5}} = \frac{T_{L_D} - IL}{T_L - IL} \]
(c) During the prestress excursion period,

\[ t_{c5} < t_c < t_{c5} + \Delta t \]

\[ PL = IL + 0.22 (TL_D - IL) \log t_{c5} \]

(d) After the prestress excursion period, \( t_c > t_{c5} + \Delta t \)

\[ PL = IL + 0.22 (TL - IL) \log t_c \]

In the above procedure, steps 1, 2, and 4(d) are the same as previously recommended in Project 66-17\textsuperscript{6}, and are represented by the line AE in Fig. 17. Steps 3 and 4(a) are simple extensions of the previous method, and are reflected by the line segment ABC, where BC is the prestress discontinuity at time \( t_{c5} \). The proposed simplification is shown as line segment BD, and calculated by Steps 4(b) and 4(c). Alternately, PL for \( t_c > t_{c5} \) may be evaluated simply as the larger of the two values given by steps 4(b) and 4(c), and \( \Delta t \) needs not be determined.
5. SUMMARY AND CONCLUSIONS

The most important conclusion of the investigation reported herein is that the prestress loss prediction procedure, developed previously in Project 66-17, can be applied to structural members in outdoors environment and generate reasonably satisfactory results. A refinement in the concrete characteristics equation (Eq. 4-14) corrects a slight inconsistency in the creep behavior, and results in improved agreement between predicted and observed long term concrete strain values. Such a refinement requires significantly increased amount of computations and is recommended only for computerized operations. When manual operations are intended, a simplified approximation is recommended, where the prestress is treated as remaining constant for a period of time following the application of loading.

In addition to the above, the reported research work also yielded the conclusions and observations listed in the following:

1. The linear distribution of concrete fiber strains across the depth of the members, and the nearly linear semi-logarithmic growth of strain with time, were confirmed.

2. The use of 1/2 in. strands in the field study did not materially effect the accuracy of the predictions.

3. Members containing stabilized strands suffered somewhat lower loss of prestress than those containing stress-relieved strands.
Because of the high level of prestress in the specimens, this difference was not large.

4. The transfer length for the 1/2 in. strands is approximately 30 in., in agreement with the current recommendations of the AASHTO as well as ACI Specifications.

5. The loss of steel tension during the curing period due to elevated temperature was nearly completely recovered after cooling to normal temperature. No lasting effect of the excursion to high temperature was observed.

6. The specimens stored outdoors exhibited more fluctuation in concrete strain, corresponding to the varying weather environment, and in general yielded lower total strain in comparison with the specimens stored indoors. However, the prediction was not improved materially by reducing the shrinkage component of the concrete characteristic relationship.

7. The differential shrinkage of the bridge deck and the precast beams did not seriously affect the usefulness of the prediction procedure. Indeed, even in its original unimproved form, the procedure yielded acceptable predictions of prestress losses. However, a generalized conclusion is not warranted at this time pending additional experimental data.
6. ACKNOWLEDGMENT

The research work reported herein is conducted at Fritz Engineering Laboratory of Lehigh University under the financial sponsorship of the Pennsylvania Department of Transportation, the U. S. Federal Highway Administration, and the Reinforced Concrete Research Council. The interest and support of these agencies are gratefully acknowledged.

Special thanks are due the staff of PennDOT Bridge Division for the design of the experimental bridge, the Schuylkill Products, Inc., for the fabrication of the pretensioned beam members and supporting specimens, the researchers of PennDOT Projects 71-7 and 71-8 at the Pennsylvania State University for their cooperation and understanding, and the research coordinator at the PennDOT Bureau of Materials, Testing and Research for the liaison among the several projects being carried out simultaneously at the bridge site.

The author acknowledges the contribution of the many research assistants who worked on different phases of this project, and the assistance of the supporting personnel at Fritz Laboratory for the conduct of the work and the production of this report.

-46-
7. **TABLES**
<table>
<thead>
<tr>
<th>Specimens</th>
<th>Strand Type</th>
<th>Storage</th>
<th>Prestress</th>
<th>Deck</th>
<th>Position on Bridge</th>
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<td>Table 2 Coefficients for Steel Surfaces</td>
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<td>Relationship</td>
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\[
\begin{align*}
\text{Instantaneous} & : A_1 = -0.04229, A_2 = 1.21952, A_3 = -0.17827 \\
\text{Stress-Strain} & : B_1 = -0.05867, B_2 = 0.00023, B_3 = 0.11860, B_4 = 0.04858 \\
\text{Relaxation Coefficients} & : B_1 = 0.00609, B_2 = 0.03245, B_3 = 0.01395
\end{align*}
\]
### TABLE 3 COEFFICIENTS FOR CONCRETE SURFACES

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<th>Coefficients</th>
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<th>Lower Bound Losses</th>
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<td>Elastic Strain $C_1^*$</td>
<td>0.02500</td>
<td>0.02105</td>
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<td>Shrinkage $D_2$</td>
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<td>-0.00066</td>
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<tr>
<td>$D_2$</td>
<td>0.02454</td>
<td>0.01500</td>
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<td>$E_1$</td>
<td>-0.01280</td>
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<td>$E_2$</td>
<td>0.00675</td>
<td>-0.00331</td>
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<td>Creep $E_3$</td>
<td>-0.00060</td>
<td>-0.00371</td>
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<tr>
<td>$E_4$</td>
<td>0.01609</td>
<td>0.01409</td>
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</tbody>
</table>

* Note: $C_1 = 100/E_c$ where $E_c$ is modulus of elasticity for concrete, in ksi
8. FIGURES
Fig. 1 Plan of Experimental Bridge
Fig. 2 Typical Cross-Section of Experimental Bridge
Fig. 3 Framing Plane of Main Beams
Fig. 4 Growth of Concrete Strains, Shrinkage Specimens
Fig. 5 Growth of Concrete Strains, Short Prestressed Specimens
Fig. 6 Growth of Concrete Strains, Beam Specimens
Beam No. 5
Stabilized Strands

Note:
Deck was completed at 168 days
Deck was completed at 173 days

Beam No. II
Stress-relieved Strands

Note:
Deck was completed at 173 days

Fig. 8 Vertical Distribution of Concrete Strains, Beam No. 11
Fig. 9 Sign Convention for Applied Loads
Stabilized Strands

Stress Relieved Strands

Fig. 10 Predicted Long Term Concrete Strains at c.g.s., Beam Specimens
Fig. 11 Predicted Long Term Concrete Strains at c.g.s., Short Pretensioned Specimens
Fig. 12 Predicted vs. Observed Concrete Strains, Specimens P1 and P2
Fig. 13 Predicted vs. Observed Concrete Strains, Specimens P3 and P4
Fig. 14 Predicted vs. Observed Concrete Strains, Specimens 3 and 11
Fig. 15 Predicted vs. Observed Concrete Strains, Specimens 4, 5, 9 and 10
Fig. 16 Refinement of Prediction Procedure

- PRESHRESS LOSS, ksi
- TIME AFTER TRANSFER, days
- Stress-Relieved Strands
- Member Weight Only
- Full Dead Load
- Refined Procedure
Fig. 17 Simplified Prediction Procedure
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