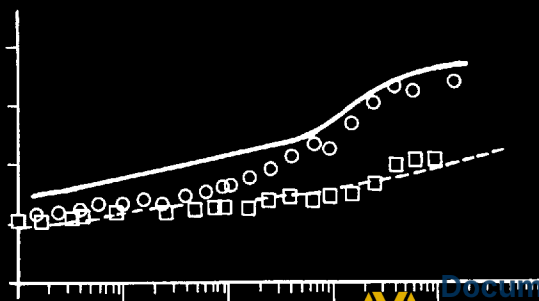
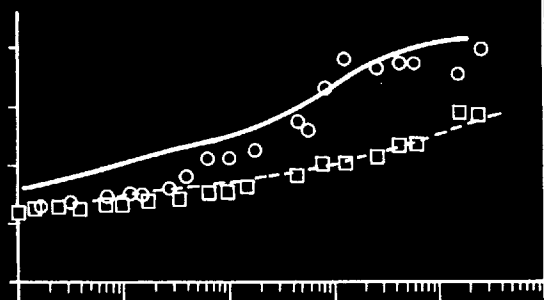
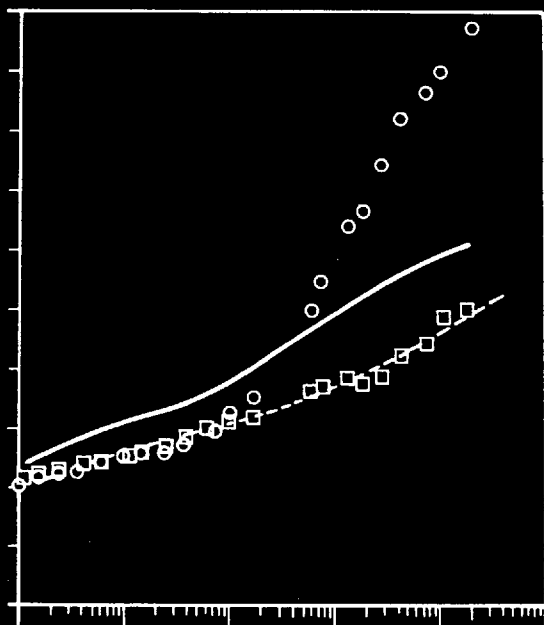


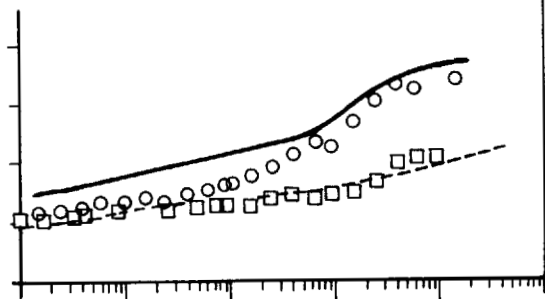
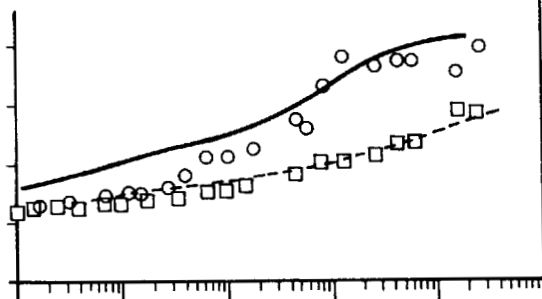
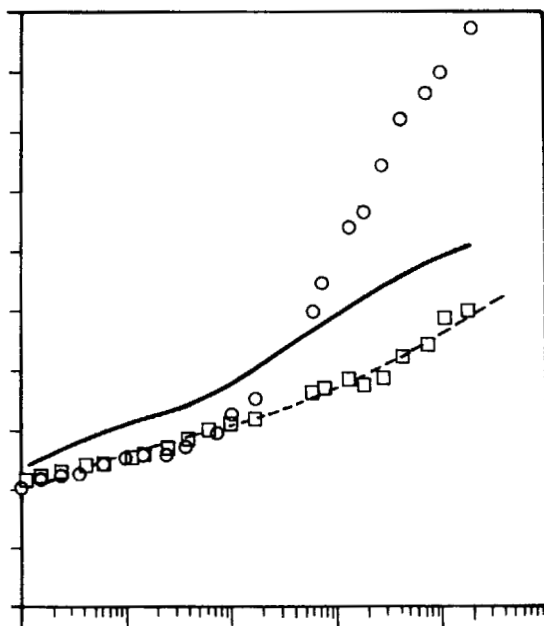
# Computer Analysis of the Effects of Creep, Shrinkage, and Temperature Changes on Concrete Structures



SP-129 

C.C. Fu and M.D. Daye  
Editors

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*Editors*



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## PREFACE

Since the ACI Special Publication SP-76, "Design for Creep and Shrinkage in Concrete Structure," was published in 1982, the research and the number of publications on this subject have reached another peak over the last decade. With today's powerful computers and sophisticated testing methods, new formulation for predicting the structural response of concrete structures to creep and shrinkage of concrete are emerging. Actual structural behavior can now be predicted by mathematical modeling of material behavior.

In an effort to keep up with these technological advancements and be able to present the engineering community with a compilation of the latest work on the subject of creep and shrinkage in concrete, ACI Committee 209 had arranged for, and sponsored two technical sessions on the subject. Members of subcommittee I took charge of organizing the sessions.

This special publication provides the means for better understanding of the important creep and shrinkage characteristics of concrete. The volume includes the papers presented at the ACI Convention held February 20 through 23, 1989, in Atlanta, Georgia. Two sessions titled "Creep, Shrinkage, and Temperature Change and Their Effects on Concrete Structures -- Computer Analysis," sponsored by ACI Committee 209, were held on Wednesday, February 22, 1989. Thirteen (13) papers by authors from the United States, Australia, Belgium, Canada and Taiwan, R.O.C., were presented in these two sessions. The authors were all requested to send their presented papers for this special publication. However, since three of the papers were not in ready-to-be-published form, they could not be included.

The authors of the papers have covered a variety of topics. Chern and his associates, from National Taiwan University, R.O.C., contributed two papers. Their papers developed and explained the theoretical and experimental parts of the long term behavior of a railway bridge. Mickleborough and Gilbert, from the University of New South Wales, Kensington, Australia, also contributed two papers about their experimental results for creep in reinforced and prestressed concrete columns.

Besides these four papers, seven authors covered other interesting topics. In Part I, Espion and Halleux, from the University of Brussels, Belgium, present their findings on long term behavior of prestressed concrete

beams. Rangan, from the University of New South Wales, Kensington, Australia, uses the current Australian Code to evaluate the creep and shrinkage deflection of reinforced concrete members. His work should be of interest to those performing comparison of different practices.

In the second session, Aswad, from Penn State University, Harrisburg, PA, U.S.A., illustrates and explains the rational and approximate methods for time-dependent deflection of prestressed concrete members. Meyers and Daye, from Bechtel Power Company, Gaithersburg, MD, U.S.A., show their practice in predicting and testing for creep and shrinkage and the approach for incorporating the results in the design of prestressed concrete nuclear reactor containment buildings. With intensive step-by-step numerical calculation, Zarghamee and Heger, from Simpson, Gumpertz and Heger, Inc., and Dana, from Ameron, present a powerful way of computing stresses and strains for each state of prestressed concrete pipes. Last, but not least, Joo and Tadros, from the University of Nebraska-Lincoln, conducted the time-dependent analysis for partially prestressed composite members.

In all, the authors have presented the state-of-the-art advancement in the field of creep and shrinkage in concrete and their effect on structural behavior.

The editors and members of ACI Committee 209 give special thanks to all the authors for their valuable contributions which made both the technical sessions and this publication possible.

C.C. Fu and M.D. Daye  
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SP 129-1

# Long Term Behavior of a Composite Prestressed Concrete Railway Bridge: Part I—Experiment

by J. C. Chern and Y. G. Wu

Synopsis: In modern computerized structural analysis, realistic material laws should be used. This research will present a constitutive law and a numerical procedure based on the finite element method for the analysis of a prestressed concrete structure including the time-dependent effects due to the load history, creep, shrinkage, aging of concrete, and relaxation of prestress. A 32.1 meter (105 ft) long U-shape railway bridge, composed of two precast post-tensioned concrete girders and a in situ cast prestressed young concrete slab, was instrumented to observe its long-term structural behavior and used for the comparisons with numerical analysis. In order to evaluate and predict the structural behavior of this concrete structure, the related experiments were designed and performed both in the field and laboratory. Some material properties needed for the analysis were obtained through the extensive program carried out in the laboratory with controlled environments. This paper will describe the details of structure, test program and experimental results. Derivations of constitutive laws and verifications by test results with numerical analysis will be shown in the Part II which follows.

Keywords: Creep properties; humidity; prestressed concrete; railroad bridges; shrinkage; strains; structural analysis.

## 2 Chern and Wu

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### INTRODUCTION

In a long-term analysis of concrete structure, it is necessary to take into account the time-dependent deformations and cracking [1,2,3]. As prestressed concrete structure exposed to the drying environment, shrinkage due to the gradual loss of moisture and creep due the sustained load can cause significant influence on its structural performance. As is well known, creep and shrinkage produce or relax stresses which cause cracking, and cracking in turn changes the stress state which causes creep.

The Da-Chia railway bridge is a simply supported prestressed concrete bridge located in Taichung county of central Taiwan. This bridge was built in 1987 with a total length of 1252 meters (4107.6 ft) and in 39 spans. As shown in Fig. 1, the cross section of bridge is composed of two precast post-tensioned prestressed girders and an in situ cast prestressed concrete slab. This type of bridge is widely adopted in Taiwan to replace some existing old steel bridges. The great advantages of using this type of bridge are manifold, such as the increase of bottom clearance of bridge providing more outlet for the flood, noise reduction due to the shielding effect of both girders, higher stiffness, lower maintenance and construction cost etc. Due to the new adoption of this composite prestressed design concept [4], the great concerns raised regarding its load resistance of composite design section and long-term behaviors including the stress redistribution between composite members and the prestress loss due to creep and shrinkage effects. A field load test was performed to validate the load carrying capacity and check the composite effect basing on the comparison of the designed position of neutral axis with that obtained by field load test [5]. This research aims only to report the portion of research which investigates the long-term behavior of this structure. In order to evaluate and predict the time-dependent structural behavior of this concrete, the assessments including two parts which are (i) the development of constitutive laws and numerical analysis to compare with test data, and (ii) the designed tests to obtain basic material data and field tests with some instrumentations on bridge. The result of first part will be presented in the second part of this paper [6]; this paper provides the results of ex-

periments planned to obtain the basic informations to assist the analysis. A flow chart which describe how the problem was approached is given in Fig. 2.

The experiments were designed to be performed both in the field and in the laboratory. The instrumentation in the field includes demountable Demec gauges for surface strain in various locations of bridge, and accompanied reinforced or plain concrete blocks designed for the measurements of shrinkage strain of bridge concrete. The extensive materials program performed in the laboratory was carried out with controlled environment. The specimens were cast simultaneously on site with the bridge and delivered to the laboratory for the material tests. The program includes shrinkage test, measurement of humidity versus time history in concrete blocks, tests for compressive strength and elastic modulus, and creep tests performed at fog or drying conditions and controlled temperature. The details about the bridge and the results of test program will be presented in the followings.

## RESEARCH SIGNIFICANCE

This research proposed a method to predict the time-dependent behavior of prestressed concrete structure; the prediction was verified with the measured field data. The satisfactory results achieved in this study indicated that the proposed method can help to predict the long-term structural response with the well planned material tests, a constitutive law which can reflect the realistic behavior, and a numerical procedure based on the formulation of finite element method.

## DESCRIPTION AND CONSTRUCTION OF TA-CHIA BRIDGE

### Description of the Test Structure

The test span of Da-Chia railway bridge is 31.3 meter long and was made by two precast girders and an in situ cast slab. The girders and slab were composited together by tranverse prestress force applied at proper intervals along the bridge. Fig. 1 shows the central cross-section and location of prestressed tendons of bridge. Some reinforcements were added to the slab and girders as shown in Fig. 3. The dimensions of both ends of girders and slab were slightly thickened to increase the shear force resistance capacity. The cross-section shown in Fig. 1 is a typical section for the structure. The design criterion for live load was done according to Japan KS-18 for railway loading. Fig. 4 shows the elevation of a girder with the locations of prestressed strands and the location of strands for slab as well. Some non-prestressing steel and prestressing bars were used as temperature reinforcement and to increase shear resistance capacity of girder respectively.

### Construction of the Bridge

A chronology of the construction of bridge is indicated in the abscissa in Fig. 11. The beginning time of the origin in this figure starts since the casting of girder A. Two girders were manufactured at the site. The girders were cast in steel mold and were demolded after twelve days of moist curing. First phase of prestressing forces including five strands (C1 - C5), shown in Fig. 4, were tensioned immediately after demolding. The prestressing force were applied from both ends of girders. Hydraulic ram and pressure gauge, which were previously calibrated by the materials testing laboratory of National Taiwan University, were used to control the pretensioning at both ends of girder. Friction test of tendons was also performed in the same time. After approximately two months since the casting of girders, the girder was lifted to be placed between two piers. The slab was then cast soon after the arrangements of formwork, reinforcement, and strands which go through the ducts. The formwork was all wood and was supported entirely from the girders by means of steel frames; steel rods tied to steel frames was arranged to go through the drainage holes of slab at proper intervals. One thirds of tranverse prestressing force to tie up the girders and slab were applied prior to the application of second phase prestressing forces, which include two strands (D1, D2) of each girder and longitudinal strands (D3 - D6) as shown in the cross-section view in Fig. 1. Two thirds of tranverse prestressing forces were applied finally. The tranverse prestressing tendons used to tie up the components was spaced at 300 mm (12 in) intervals along the girder. After the application of prestressing forces at each stage, the prestressing ducts was grouted with nonshrink mortar.

### Materials Used in Bridge

#### Concrete

The concrete used to cast the bridge were made with the mix shown in Table 1. ASTM Type I portland cement and crushed gravel from Ta-Chia river were used for the mix. Sieve analyses for coarse and fine aggregate are listed in Table 2. The average modulus of elasticity and compressive strength are listed in Table 3. Samples of concrete specimens used in strength, and also for creep and shrinkage tests were taken during the casting of concrete.

#### Reinforcement

The prestressing reinforcement is 7 mm diameter 12-wire high tension strand. For prestressing tendons C1 - C5 are 11 strands wired together in the same duct (denoted as 11T12-7 mm  $\phi$ ); while tendons D1 - D6 are 12 strands in each duct (12T12-7 mm  $\phi$ ). Transverse prestressed tendons are four strands in the duct (4T12-7 mm  $\phi$ ). The specified minimum breaking strength of strand is 270ksi (19,000 kg/cm<sup>2</sup>); the average value of breaking stress and yield stress from test samples are 273.9 ksi and 248.7 ksi respectively. Young's modulus is  $28.4 \times 10^6$  psi.

Tests of non-prestressing reinforcement show that #5 bar (5/8 in) has an average yield strength 45 ksi and ultimate tensile strength 65 ksi using three samples; #4 bar (1/2 in) shows higher test values which are 58.3 ksi and 86.7 ksi respectively.

## TEST PROGRAM

The test program was composed of two parts including tests done in the field and in the laboratory. The details of the planned test are described herein.

### Shrinkage Test

The shrinkage tests conducted at drying room which was automatically controlled to maintain constant temperature ( $23 \pm 0.5^\circ\text{C}$ ) and constant humidity ( $50 \pm 2\%RH$ ). Specimens with  $4 \times 4 \times 14$  in and  $3 \times 3 \times 12$  in prisms were used for the tests. The other test variable is the age when concrete specimens were exposed to drying environment. Each test set included four specimens, two were specimens for drying test and two other specimens remained in fog room and acted as control specimens. The test was carried out according to ASTM C341-84; ELE Demec strain gauge was used to measure the strain of specimens at proper time intervals. The gauge length of each pair of mechanical gauges is 8 in (200 mm). The effective drying shrinkage for all specimens was counted as the difference between the amount of shrinkage measured in dry room and that of respective companion specimen.

### Creep Test

Creep tests were performed at moist or drying condition at the laboratory. All specimens were  $6 \times 12$  in cylinders and were unsealed. Basic creep test was conducted in the moist room (fog room) at controlled  $23^\circ\text{C}$  temperature; the other test series was performed in the drying room at 50% relative humidity and  $23^\circ\text{C}$  temperature. Specimens were placed in the curing room until the age when the load was applied. The loading age of concrete has great effect on the development of creep of concrete [7,8]. The loading ages of concrete specimens were designed to be 7, 29 and 94 days. Test series performed at fog room, called as basic creep, was used to find the basic informations regarding the viscoelastic aging behavior of bridge concrete.

During the test, the creep test specimens were mounted on the creep loading frames which were composed of a 44.8 k jack, a load cell and a system of coil springs held in compression by a system of rods and plates between which the specimen was clamped. The desired load, which is roughly calculated as one fifth of the strength measured from companion strength cylinders, was applied by a hydraulic jack [Table 4]. After loading in the frame, the nuts on the tension rods of the load-sustaining apparatus were brought up against the end plates. The actual sustained load, which is listed in Table 4, was monitored by the PC control data acquisition system with the signals passing from load cell. The details of testing procedures of using these creep apparatus is given in Ref. 9. There were control specimens stored with no load in the moist room as dummy specimens and strength cylinders used to provide the information such as modulus of elasticity and compressive strength.

### Internal Humidity of Concrete

Six concrete blocks were cast to investigate the behaviors of concrete including the internal humidity, temperature history due to hydration, shrinkage, and the effect of reinforcement to restrain shrinkage. This paper only presents the result of humidity measurement, other test results were described in Ref. 10.

These concrete blocks, with dimension  $16 \times 16 \times 14$  in, were cast in field using bridge concrete. Specimens were sealed from five sides with insulating polystyrene foam to prevent the heat exchange and plastic sheets to avoid the moisture exchange with the ambient. The purpose for allowing only one exposure surface was aimed to simulate the real exposure condition of bridge slab, which has only one exposure surface after the completion of the waterproofing work on the top of bridge slab. Two specimens were drilled to different depth to allow the installation of sleeves for the measurement of humidity in concrete [Fig. 5]. The newly developed sleeves can hold tightly to the concrete surface and guarantee the correct measurement at specific depth of concrete using the double seals design [Fig. 5]. The specific depths for the humidity measurement are 2.3, 4.5, 9.5, 13, and 20 cm from the exposure surface of concrete block. During the time of measurement, one simply pulls the peg out of the hole and insert the Vaisala HMP 32 UT measuring probe. The accuracy of measurement for humidity is  $\pm 2\%$  RH at  $20^\circ\text{C}$ .

### Surface Strain of Field Bridge

Several mechanical gauge points similar to that used in Ref. 11 were installed on the bridge girders and slab soon after the demolding of form work. Gauge point lines were embedded at locations near the ends, quarter and center span of bridge. For each gauge point line, gauge points were located at top, head, web, and bottom of the girder cross-section [Fig. 11]. The strains were measured by Demec strain gauge.

## EXPERIMENTAL RESULTS AND DISCUSSIONS

The test results of all series conducted in this study are given and discussed as follows.

### Shrinkage test

The results of shrinkage test are given as the average of two shrinkage specimens dried together. For each shrinkage specimen, the shrinkage strain was obtained by the averaging of measured strains on two opposite surfaces. Fig. 6 shows the test results for shrinkage specimens with two sizes; the faster drying which occurs in the smaller specimens results in higher shrinkage strain. Fig. 7 gives the shrinkage strain for specimens started tests at various ages; it shows that specimens with an earlier exposure time yields a higher shrinkage strain. This fact can be explained through the following two reasons: firstly, the aging effect on the coefficient of diffusion resulted in slower moisture diffusion and less shrinkage in the well cured old concrete as explained in Ref. 13; secondly, the increase

of stiffness due to the aging effect resists the shrinkage of concrete more effectively.

### Creep Test

The main variables in the creep series were the loading age and the testing environments. The results are given as the average of two creep specimens subjected to the same sustained load. For each creep cylinder, the creep strain was the averaging strain of three mechanical gauge point sets located on the surface of specimen. Creep strain obtained in this test was counted as the difference between measured strain of creep specimen and its companion specimen. Fig. 8, giving the results of tests, shows the effect of age of concrete at time of loading on creep deformations. Three ages at loading were chosen, namely, 7 days, 29 days and 94 days. It can be seen that creep is greater for specimens loaded at younger age. This phenomenon is valid for the creep test performed either at drying room or at fog room. After eight months of loading at moist condition; the creep for the cylinders loaded at age 7 days was about 2.5 times higher than for the cylinders loaded at age 94 days. If we consider the same case except that the specimens were loaded at drying room, the creep ratio is approximate 1.7 [Fig. 8].

For the concretes loaded at the age of 7 days and 29 days, the combination of creep and shrinkage is five times the instantaneous deformations; for specimens loaded at 94 days this value is 4.3 [Fig. 8].  $J(t, t')$ , the ordinate of Fig. 8, is called as creep function, which represents the strain at time  $t$  caused by a unit sustained stress acting since time  $t'$ .

Fig. 9 gives the results of drying creep strain per unit stress versus various ages of loading. It can be seen that the drying creep for concrete loaded at younger age is much higher than that for the specimens loaded at older ages. After eight months of loading at dry environment, the drying creep for the cylinders loaded at age 7 days was about 2.5 times higher than that for the cylinders loaded at age 94 days. Drying creep, a well known phenomenon called as Pickett effect [12], was recently explained by a mechanism developed by Bažant and Chern [9]. From the microstructural viewpoint, their model rests on one simple hypothesis, namely, that there exists some mechanism causing the viscosity coefficients which characterize the creep rate to depend on the magnitude of the flux of the local microdiffusion of the water between the macropores (capillary pores) and the adjacent micropores in the cement gel (gel pores etc.). The fast moisture change results in the bond rupture of the gel structure and induces additional deformation. Therefore, a coefficient  $\bar{r}$  treated as a material constant, was introduced mainly to describe this phenomenon. It was further proposed by Ref. 9 that the coefficient  $\bar{r}$  can be a function of loading age  $t'$ . Chern et al. [9] presumed that the strength of bond of gel structure which can resist the stress due to moisture flux should develop with age. It means that the Pickett effect for concrete loaded and dried at a younger age should be more obvious than that for concrete loaded at older age. Due to the weaker bond strength and higher chemical reactive potential in the early age, active debonding and rebonding process is the cause of higher drying creep of concrete. This argument can be seen from the test result shown in Fig. 9.

### Internal Humidity of Concrete

The measurement of internal humidity of concrete can provide the basic information regarding the process of moisture loss of bridge concrete to the drying environment. The shrinkage behavior of concrete was found to be depending on the internal humidity of concrete [8,14]. The test result presented in Fig. 10, showing the distribution of internal humidity at various times, can be used to find the coefficient of diffusion and then for the analysis of shrinkage strain of concrete.

### Surface Strain of Field Bridge

The mechanical gauge points located on bridge were measured using Demec gauge at proper time intervals. Strains are plotted versus time in Figs. 11 and 12, which are measured at gauge lines of the quarter and center span of girder A. Time in horizontal axis is counted since the casting of girder A. Notations AP, BP, and PP represent the respective time to apply the prestressing force on girder A, girder B, and slab. Notations A-girder, B-girder, and Plate shown in the figures show the date of the casting of respective components of bridge. Time indicated by arrow sign and notation 'STONE' is the date when sub ballast was placed to the structure. The readings plotted in Figs. 11 and 12, are all corrected back to the temperature at the time of initial reading as what was done in Ref. 11. The coefficient of thermal expansion obtained from the test and used for calibration is  $13.5 \times 10^{-6}/^{\circ}\text{C}$  [10]. This value is quite similar to that was found by Neville [15] using same gravel aggregate concrete.

## CONCLUSIONS

This paper describes the material used for construction and the details of bridge structure; it also shows the test results which were obtained from both the field and indoor tests. The usage of these test results to analyze the long-term structural behavior of this bridge is relegated to a subsequent companion paper. Some observations found by the tests are summarized as follows:

1. The older the specimen at time of drying, the less the shrinkage; the larger the size of the specimen, the less the rate of development of shrinkage.
2. The age on loading has significant effect on the magnitude of both drying creep and basic creep. The older the specimen at time of loading, the less the basic creep and drying creep.
3. The fact of larger drying creep due to earlier loading age can well correlate to the microstructural view of the creep mechanism. The weak early bond strength and more active debonding and rebonding potential of solid particles or molecules of calcium silicate hydrate are attributed to the causes of higher drying creep for concrete loaded and dried at earlier age.

## REFERENCES

1. Anderson, C.A., "Numerical Creep Analysis of Structures," Chapter 8 in Bažant, Wittmann, Eds, *Creep and Shrinkage in Concrete Structures*, John Wiley and Sons, London, 1982, pp. 259-303.
2. Wittmann, F.H. and Roelfstra, P.E., "Total Deformation of Loaded Drying Concrete," *Cem. Concr. Res.*, Vol. 10, No. 5, 1980, pp. 601-610.
3. RILEM Committee TC69-MMC (Chaired by Z.P. Bažant), "Conclusions for Structural Analysis and for Formulation of Standard Design Recommendations on Creep and Shrinkage," *ACI Materials Journal*, Vol. 84, No. 6, 1987, pp. 578-581.
4. Kosuda, N. et al., "Design and Construction of Prestressed Concrete Railway Bridge in the City," *Prestressed Concrete*, Vol. 19, No. 6, 1977, pp. 46-53. (in Japanese).
5. Chern, C.C. et al., "Loading test and Other Related Tests on U-Type Da-Chia Railway Bridge: Part I – Loading Test," Report TR-75009a, Department of Civil Engineering, National Taiwan University, Sept. 1987, 72 pp. (in Chinese).
6. Chern, J.C. et al., "Long-Term Behavior of A Composite Prestressed Concrete Railway Bridge: Part II – Constitutive Law and Analysis," American Concrete Institute, 1989 Annual Convention, Feb. 1989, Atlanta, GA.
7. Bažant, Z.P. and Wu, S.T., "Creep and Shrinkage Law for Concrete at Variable Humidity," *Journal of Eng. Mech. Div., ASCE*, Vol. 100, No. EM6, 1974, pp. 1183-1209.
8. Bažant, Z.P. and Chern, J.C., "Concrete Creep at Variable Humidity: Constitutive Law and Mechanism," *Materials and Structures (RILEM, Paris)*, Vol. 18, No. 103, 1985, pp. 1-20.
9. Chern, J.C., Wu, Y.G., and Chang, H.C., "The Influence of Loading Age on Long-Term Drying Creep of Concrete," *Journal of the Chinese Institute of Engineers*, Vol. 11, No. 2, 1988, pp. 113-120.
10. Chern, J.C. et al., "Loading test and Other Related Tests on U-Type Da-Chia Railway Bridge: Part II – Creep, Shrinkage, Shear Transfer, and Numerical Analysis," Report TR-75009b, Department of Civil Engineering, National Taiwan University, Sept. 1987, 176 pp. (in Chinese).
11. Gamble, W.L., "Long-Term Behavior of A Prestressed I-Girder Highway Bridge in Champaign County, Illinois," Report UILU-ENG-79-2019, University of Illinois at Urbana-Champaign, Dec. 1979, 101 pp.
12. Pickett, G., "The Effect of Change in Moisture Content on the Creep of Concrete under a Sustained Load," *J. of Amer. Concr. Inst.*, Vol. 38, 1942, pp. 335-355.

13. Bažant, Z.P., "Mathematical Models for Creep and Shrinkage of Concrete," Chapter 7 in Bažant, Wittmann, Eds, Creep and Shrinkage in Concrete Structures, John Wiley and Sons, London, 1982, pp. 163-258.
14. Bažant, Z.P. and Najjar, L.J., "Nonlinear Water Diffusion in Nonsaturated Concrete," Materials and Structures, (RILEM, Paris), Vol. 5, 1972, pp. 3-20.
15. Neville, A.M., "Properties of Concrete," Chapter 7, Pitman Publishing Limited, London, 1981, pp. 496.

TABLE 1 -- CONCRETE MIX INFORMATION

| Slump<br>cm | W/C<br>% | S/A<br>% | air<br>% | unit weight $kg/m^3$ |        |            |              |
|-------------|----------|----------|----------|----------------------|--------|------------|--------------|
|             |          |          |          | water                | cement | fine aggr. | coarse aggr. |
| 8           | 42.5     | 39.1     | 1.5      | 190                  | 447    | 669        | 1042         |

\*  $1 \text{ kg}/m^3 = 1.71 \text{ lb}/yd^3$ ,  $1 \text{ cm} = 0.394 \text{ in.}$

TABLE 2 -- SIEVE ANALYSES FOR COARSE AGGREGATE AND FINE AGGREGATE

| Sieve size                         | $1\frac{1}{2}"$ | 1"     | $\frac{3}{4}"$ | $\frac{1}{2}"$ | $\frac{3}{8}"$ | #4   | #8      |
|------------------------------------|-----------------|--------|----------------|----------------|----------------|------|---------|
| Cummulative percentage passing (%) | 100             | 95     | 86             | 60             | 18             | 1.4  | 0       |
| Sieve size                         | #4              | #8     | #16            | #30            | #50            | #100 | F.M.    |
| Cummulative percentage passing (%) | 100             | 89     | 72             | 53             | 31             | 9.6  | 2.45    |
| ASTM C33                           | 95-100          | 80-100 | 50-85          | 25-60          | 10-30          | 2-10 | 2.5-3.1 |

TABLE 3 -- MODULI OF ELASTICITY AND COMPRESSIVE STRENGTH

| Age of test<br>(days) | cylinder strength*    |                   | modulus of elasticity<br>$E_c(10^5 \text{ kg/cm}^2)^{**}$ |
|-----------------------|-----------------------|-------------------|-----------------------------------------------------------|
|                       | $f_c(\text{kg/cm}^2)$ | $f_c(\text{psi})$ |                                                           |
| 7                     | 277.3                 | 3934              | 2.24                                                      |
| 28                    | 290.6                 | 4130              | 2.57                                                      |
| 29                    | 313.6                 | 4451              | 2.71                                                      |
| 94                    | 375.3                 | 5327              | 2.77                                                      |

\* average value of 1 to 4 specimens.

\*\*  $1 \text{ kg/cm}^2 = 14.2 \text{ psi}$ 

TABLE 4 -- DETAILS OF AGE LOADING AND APPLIED STRESS FOR CREEP TEST

| Specimen No. | Age of loading<br>(days) | Applied stress<br>( $\text{kg/cm}^2$ )* | Test environment             |
|--------------|--------------------------|-----------------------------------------|------------------------------|
| C 7-1,2      | 7                        | 53.8                                    | fog room, $23^\circ\text{C}$ |
| D 7-1,2      | 7                        | 54.7                                    | 50% RH, $23^\circ\text{C}$   |
| C 29-1,2     | 29                       | 67.5                                    | fog room, $23^\circ\text{C}$ |
| D 29-1,2     | 29                       | 56.6                                    | 50% RH, $23^\circ\text{C}$   |
| C 94-1,2     | 94                       | 60.9                                    | fog room, $23^\circ\text{C}$ |
| D 94-1,2     | 94                       | 57.7                                    | 50% RH, $23^\circ\text{C}$   |

\*  $1 \text{ kg/cm}^2 = 14.2 \text{ psi}$

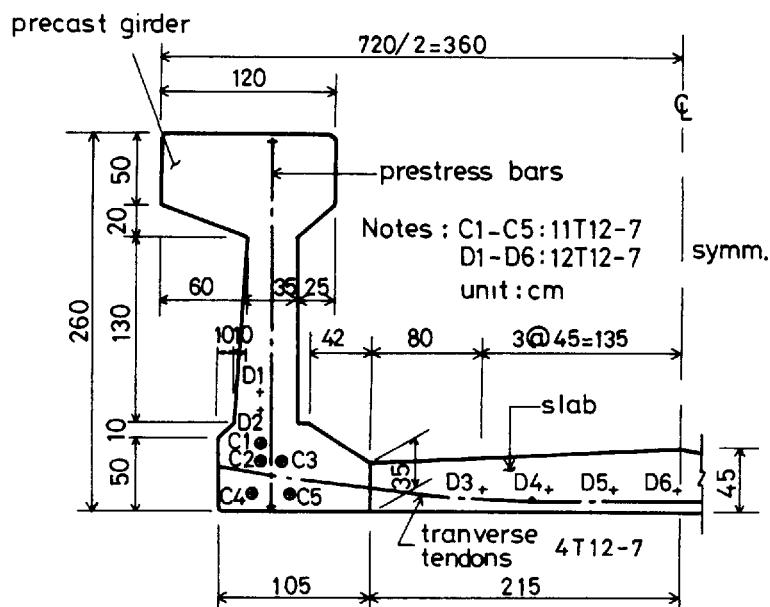


Fig. 1--Cross section of bridge and arrangement of prestress tendons and bars ( 1 cm = 0.394 in.)

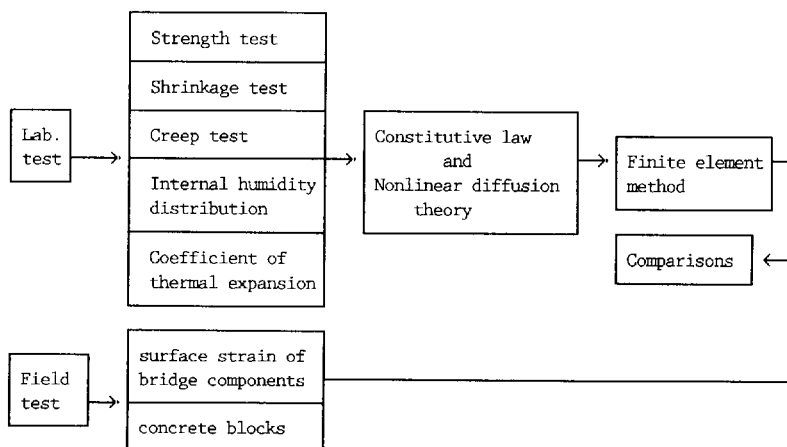


Fig. 2--Flow chart showing the content of research and its sequence

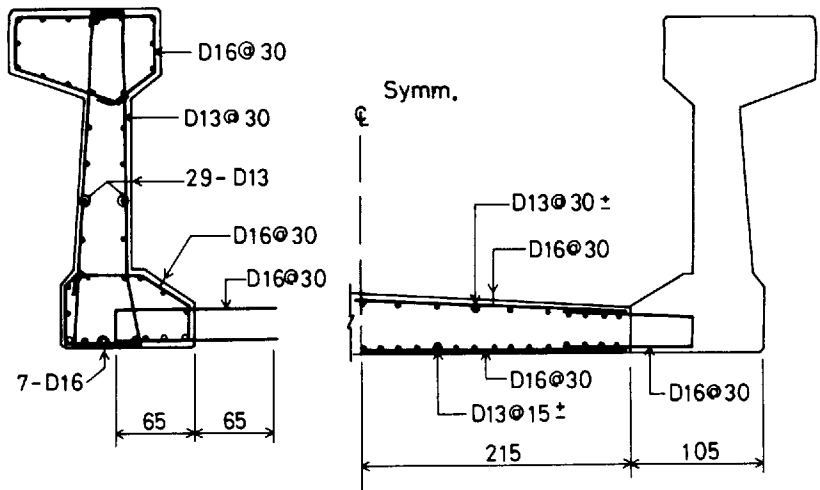


Fig. 3--Girder and slab cross section, showing reinforcement

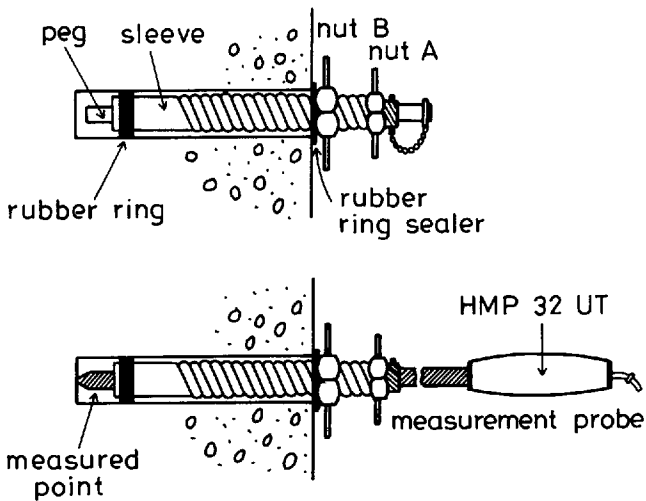


Fig. 5--Measurement of internal humidity in concrete

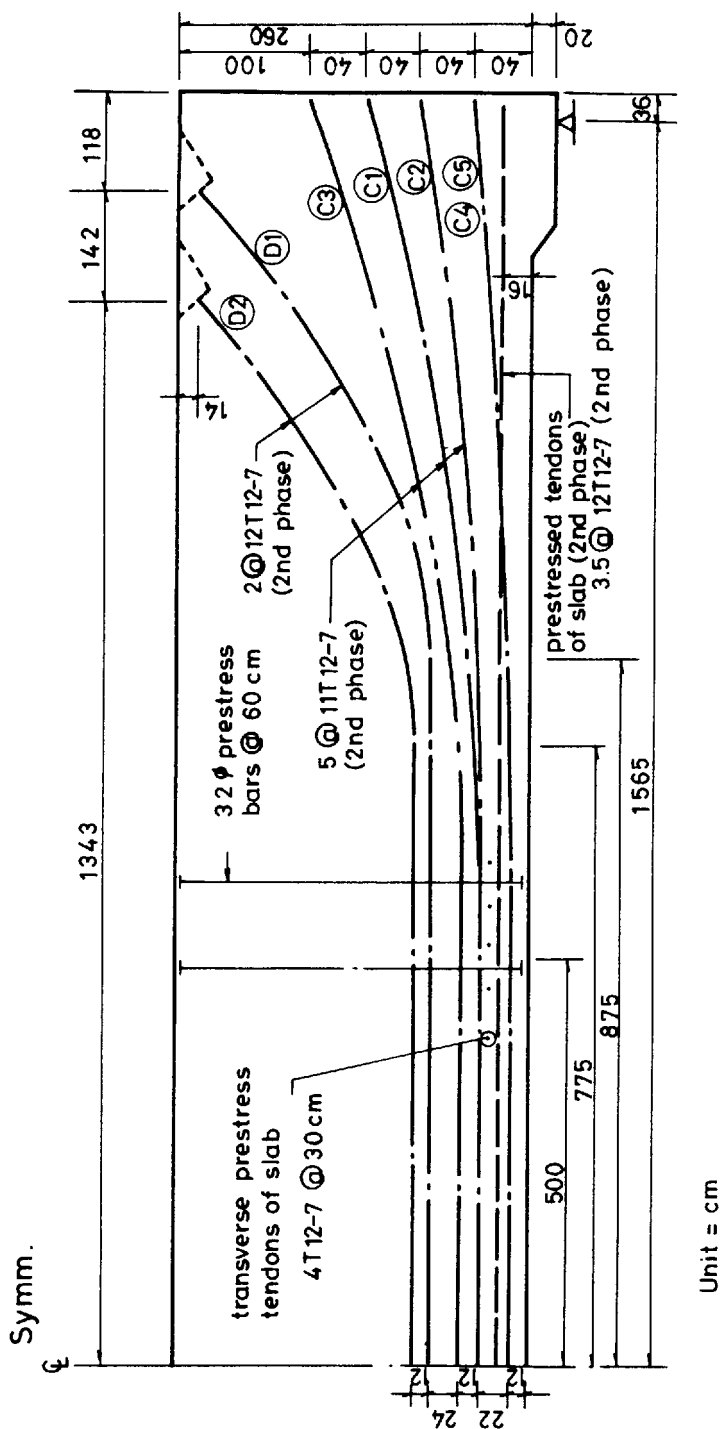
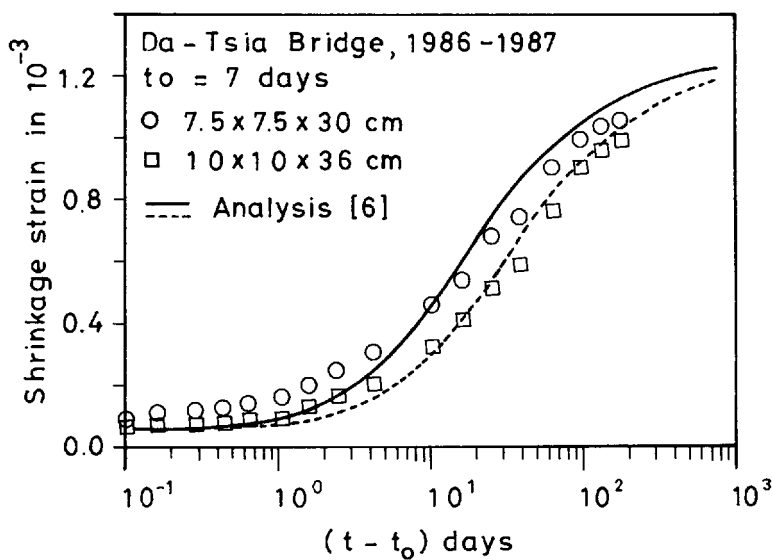
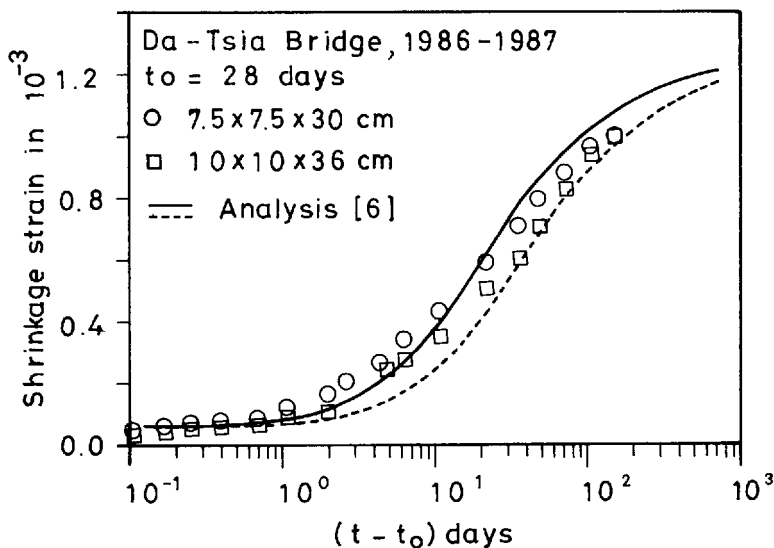


Fig. 4--Elevation of girder showing the arrangement of prestress tendons and bars



a)



b)

Fig. 6--Shrinkage strain versus drying time for specimens with two sizes: a)  $t_0 = 7$  days; b)  $t_0 = 28$  days

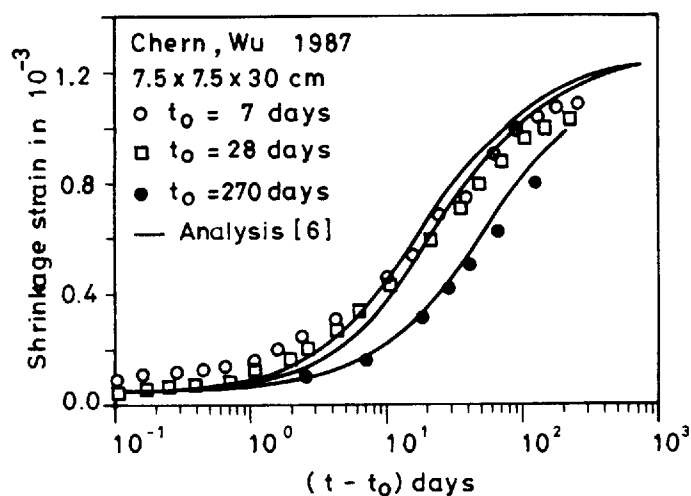


Fig. 7--Shrinkage strain versus drying time for concrete beginning drying at various stages

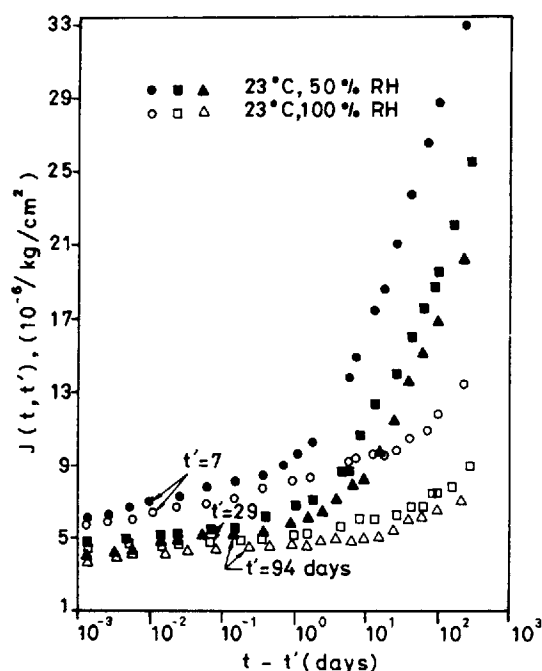


Fig. 8--Creep compliance versus time after loading for concretes tested at the fog room and drying room

Fig. 9--Drying creep compliance of concrete versus time after simultaneous drying and loading

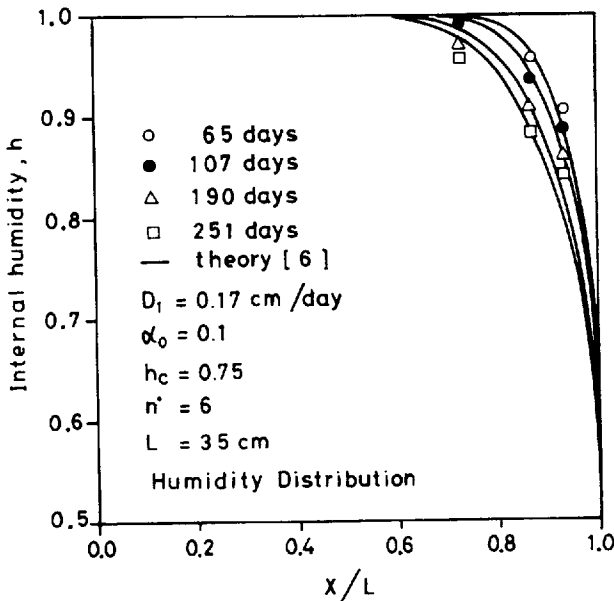
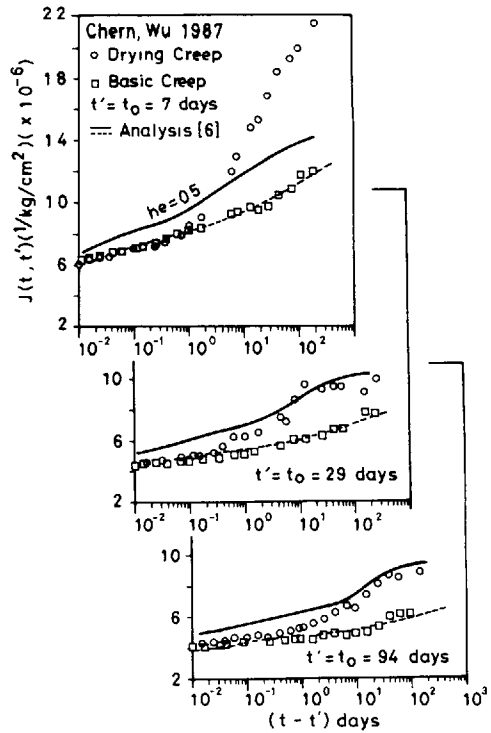


Fig.10--Distribution of internal humidity of concrete block

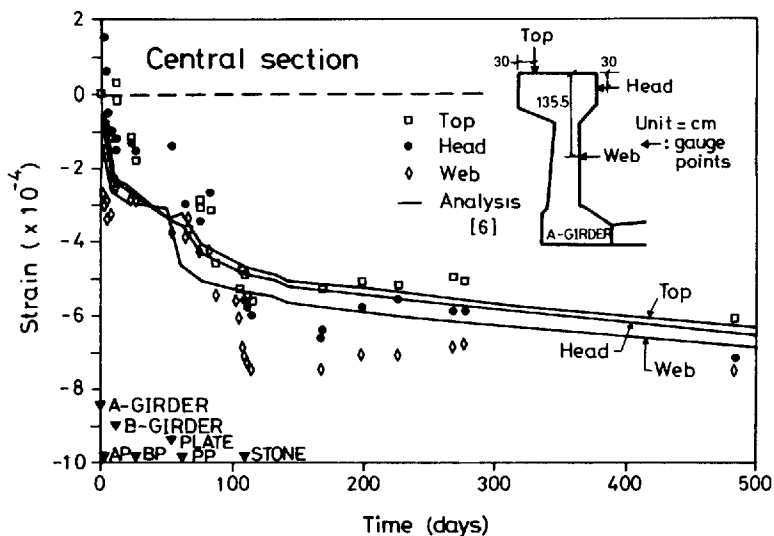


Fig. 11--Strain-time curves at midspan of girder A

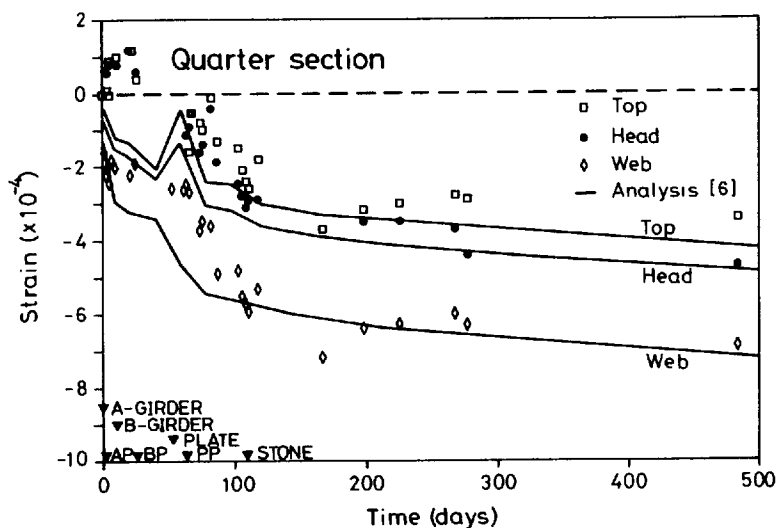


Fig. 12--Strain-time curves at quarter span of girder A

SP 129-2

# Long Term Behavior of Prestressed and Partially Prestressed Concrete Beams: Experimental and Numerical Results

by B. Espion and P. Halleux

Synopsis : Tests results from long-term experiments on prestressed and partially prestressed concrete beams are reported. Tests were carried out on 10 rectangular beams spanning 8 m and undergoing sustained loading for five years. After that time, there was no evidence of stabilization of the time-dependent behavior of concrete. Numerical modeling of the deformation of the midspan section explains experimental observations and confirms that the presence of ordinary reinforcing steel in a prestressed concrete section leads to a redistribution of stresses between concrete and steel which should be taken into account in serviceability limit states computations. Tests to failure of the beams at five years yield no significant differences in carrying capacity with tests executed at an early age. It is suggested that the deflection limit state is a major consideration in design and that the degree of prestressing should be chosen in function of ratio of permanent load to total design load (permanent and live).

Keywords: Beams (supports); camber; cracking (fracturing); creep properties; deflection; failure; loads (forces); partial prestressing; prestressed concrete; prestress loss; serviceability; shrinkage; strains; stress analysis; structural design.

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ACI Member Pierre Halleux is Professor and Head of the Department of Civil Engineering at the University of Brussels. Among many research interests, he has been associated since 1970 with several experimental research efforts on the behavior of reinforced, prestressed and partially prestressed concrete structures under long-term loading.

## INTRODUCTION

The concept of "structural concrete" is presently gaining popularity among concrete researchers and designers. Following that concept, reinforced, prestressed and partially prestressed concrete structures should be designed in a unified approach of limit states. Obviously, partially prestressed concrete, which combines tensioned and untensioned reinforcement becomes the general case of structural concrete. The computation of the ultimate limit states is covered by well accepted rules in all major standards or codes of practice for designing structural concrete, but the computation of the serviceability limit states is still a debated question.

Particular methods have been developed for the verification of the serviceability limit states (deflection, deformation, cracking, ...) of reinforced and fully prestressed concrete structures. They are supported by hundreds of long-term laboratory tests and in situ observations. Some analytical methods have also been proposed for the verification of the serviceability limit states of partially prestressed concrete, but well accepted design methods, covering the whole range of structural concrete, are still needed. A possible reason explaining this lack of consensus is the rather limited number of reliable long-term experiments on partially prestressed concrete beams available to evaluate the degree of accuracy of these methods.

## RESEARCH SIGNIFICANCE

Since 1975, the laboratories of civil engineering of the Belgian Universities have been continuously and commonly engaged in long-term tests on fully (PC) and partially (PPC) prestressed beams. The whole program involved the testing of 80 beams (24 short-term tests and 56 long-term tests) with span 8 m. The

purpose of the present paper is to report the main results from the tests carried out at Brussels University (ULB) and to compare them with numerical predictions.

#### EXPERIMENTAL INVESTIGATION

In this research, we define the degree of prestressing by the following index  $\lambda$  :

$$\lambda = \frac{\sum_i A_{pi} \cdot f_{y,0.2,k} \cdot d_{pi}}{\sum_i A_{pi} \cdot f_{y,0.2,k} \cdot d_{pi} + \sum_i A_{si} \cdot f_{y,k} \cdot d_{si}} \quad (1)$$

with  $A_{pi}$ , the area of prestressing reinforcement at depth  $d_{pi}$ , and  $A_{si}$ , the area of ordinary reinforcement at depth  $d_{si}$ ; the characteristic yield strength of the prestressing reinforcement is  $f_{y,0.2,k}$  (at 0.2 % offset) and  $f_{y,k}$  is the characteristic yield strength of the ordinary reinforcement.

This definition is such that the degree of prestressing  $\lambda$  equals 1 in case of full prestressing (no ordinary reinforcement). Its value becomes 0 in the case of ordinary reinforced concrete (no prestressing reinforcement). Additionally, fully prestressed concrete structures are structures in which tensile stresses are not allowed in concrete. The concrete of partially prestressed concrete structures may exhibit tensile stresses and even some cracking.

We tested beams with rectangular section ( $h = 400$  mm;  $b = 340$  mm) post-tensioned either by BBRV grouted cables (24 wires diam. 7 mm for beams with  $\lambda = 1.0$ ) or by unbonded strands (8 strands diam. 1/2" for beams with  $\lambda = 0.8$  and 5 strands diam. 1/2" for beams with  $\lambda = 0.5$ ). The tendon layout (Fig.1) is linear for the beams with  $\lambda = 0.8$  and polygonal ( $\tan \alpha = 0.056$ ) for the beams with  $\lambda = 1.0$  or 0.5. The geometrical description of the beams, the reinforcement detailing and the mechanical properties of the steel are described in Fig.1 and 2 and reported in Tables 1, 2 and 3.

The designation of the beams and the description of the loading program are given in Table 4. We have 4 "reference" short-term (ST beams) tests to failure at an early age (from 28 to 56 days). Then, we submitted 10 beams (LT) to long-term sustained loading for 4 1/2 to 5 years under loading levels varying between the dead weight to the full service load (constituted by quarter points loads  $Q$ ). At the age of 5 years, all LT beams were tested to failure.

The PC beams were designed elastically with allowable stress concepts; the intended stress distribution over the depth of the midspan section was :

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- a triangle with zero stress at the upper fibre in the case of beams submitted to their dead weight only;
  - a triangle with zero stress at the lower fibre in the case of beams submitted to their full service load;
  - a rectangle in the case of beams loaded with  $Q = Q_s/2$ .
- The service loading level corresponds to a value of  $Q_s$  equal to 63.75 kN. The PPC beams were designed at ultimate limit state to have the same service load with a nominal safety against bending failure equal to 1.5.

Actually, experiments were carried out in two phases. Tests on PC beams were executed between 1975 and 1980 under laboratory, but uncontrolled environmental conditions. Tests on PPC beams were executed between 1981 and 1986 under controlled environmental conditions : 60 % ( $\pm 5$  %) relative humidity and 20 °C ( $\pm 2$  °C).

The concrete composition was :

|             | Mix PC     | Mix PPC    |
|-------------|------------|------------|
| Gravel 4/16 | 1250 kg    |            |
| Gravel 4/14 |            | 1270 kg    |
| Sand        | 570 kg     | 580 kg     |
| Cement P40  | 375 kg     | 375 kg     |
| Water       | 180 litres | 180 litres |

Individual concrete strengths are reported in Table 5.

Along with the PPC tests, creep and shrinkage characteristics were measured on 6 prisms 150 mm  $\times$  150 mm  $\times$  600 mm. All prisms were cured for one day; 3 prisms remained unloaded (shrinkage prisms) and the 3 other prisms (creep prisms) were loaded at 14 days under different stress levels (5, 10 and 15 MPa). Fig.3 gives the time-dependent evolution of the total mean strain of these prisms up to 1000 days. The experimental shrinkage strain evolution  $\epsilon_{sh}$  and the experimental creep function  $J(t, t')$  may be fitted by the following formulae :

$$\epsilon_{sh}(t, t_{sh} = 1) = -0.0006 \frac{(t-1)}{78 + (t-1)} \quad (2)$$

$$J(t, t') = \frac{1}{26000 \text{ MPa}} \left( 1 + 2.5 \frac{(t-t')^{0.6}}{15 + (t-t')^{0.6}} \right) \quad (3)$$

with  $t$ , the age in days and  $t'$ , the age at loading in days. The various constants in Eq.2 and 3 have been obtained by linear

regression of test data. These mathematical forms have been selected here for their similarity with the ACI 209 prediction model (1). Note that the inverse of the "elastic" compliance ( $t = t'$  in Eq.3) provides a rather low value of 26000 MPa for the "elastic" modulus. This value includes some short-time creep and nonlinear effects and should not be confused with the actual value of the secant elastic modulus  $E_c$  which may be taken as 30000 MPa at any age.

No creep prisms were made with the PC beams. As a first approximation, the same time-dependent prediction model may be used for analyzing the results of the PC beams. This is justified by the following reasons :

- the two concrete compositions are close and are characterized by similar consistency values (values of the flow test : from 1.73 to 1.93);
- the concrete was mixed and cured by the same methods, in the same laboratory, and placed by the same people;
- the average environmental conditions prevailing during the PC tests were also : 60 % R.H and 20 °C.

Measurements consisted mainly in deflection and strain readings with mechanical extensometers (gage length : 250 mm) in several sections and along different fibre levels. Crack widths were also recorded. When deflection and deformation (mid-depth strain, section curvature) are mentioned in this paper, we always refer to their values in the midspan section. Values in other cross sections, and more experimental details may be found in References 2 and 3. For LT beams, we will also commonly call "unloaded" beams, the beams which were submitted to their dead weight only ( $Q = 0$ ) and "loaded" beams, the beams which sustained additional load.

## EXPERIMENTAL RESULTS

### Time Dependent Behavior

The time-dependent evolution of the midspan deflection is given in Fig.4 for the unloaded beams, and in Fig.5 for all other LT beams. The evolution of the camber of the unloaded beams is remarkable : the camber of the PC beams still increased -although at a very slow rate- after five years, whereas the camber of PPC beam LT-0.8 stabilized rapidly; the camber of PPC beam LT-0.5 diminished soon after prestressing and even became net positive deflection after 200 days. This difference in long-term behavior is typically due to stress redistribution between concrete and ordinary reinforcement in the PPC beams.

The PPC beams LT-0.8-Q and LT-0.5-Q (Fig.5) were both cracked under full service load. The pronounced sag exhibited by these two beams (especially by beam LT-0.5-Q) would render them unserviceable for most applications. These two tests were nevertheless helpful to evaluate the extent of cracking under full

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service load : the maximum crack width never exceeded 0.2 mm. The behavior of the two PC beams LT-1.0-Q/2 should also be noticed : their long-term deflection approaches zero at five years and we should remember here that the assumed stress distribution is constant over the depth of the midspan section, which roughly implies zero curvature.

The time-dependent evolution of the midspan section curvature is depicted in Fig.6 for the PC beams (LT-1.0), in Fig.7 for the PPC unloaded beams and in Fig.8 for the PPC loaded beams. A close similarity between the trends of these curves and the trends of the deflection curves should be noted. The long-term curvature of the PC beams (Fig.6) follows logically from the assumed stress distributions in these sections.

Finally, Fig.9, 10 and 11 give the time-dependent evolution of the mid-depth strain of the midspan section. The mid-depth strain of the PC beams (Fig.9) seems independent of the loading history : taking into account some inevitable shrinkage, creep and temperature scattering, all PC beams prestressed at 14 days exhibited similar mid-depth long-term strain. Furthermore, since the long-term trend may typically be fitted by a straight line in semi-log time scale, it should be deduced that the time-dependent behavior (in particular the shortening of the member) was not stabilized after five years. Naturally, the rate of evolution measured in actual time scale was continuously diminishing. The mid-depth strain exhibited by the PPC beams was evidently less pronounced. The strain "jump" in Fig.11 is a consequence of cracking and uplifting of the neutral axis position in the loaded PPC beams.

### Ultimate Tests to Failure

At the age of five years, all LT-beams were tested to failure. The failure loads and the ultimate carrying capacities are reported in Table 5 where they can be compared with the short-term values. Statistically, there is no increase, but also no decrease in the actual safety, which ranges from 2.14 to 2.71. The average safety of the PC beams is greater than the average safety of PPC beams, but also much scattered : this is probably due to a more brittle behavior at failure.

### NUMERICAL ANALYSIS

The necessity of developing mathematical models to perform the time-dependent analysis of the deformations of cracked PPC sections has been recognized for some time. Several models have already been proposed (6-13). The model used in the present analysis belongs to the same kind of approach.

Its main assumptions are :

- linear strain distribution over the depth of the section before and after cracking;

- linear stress-strain relationship for concrete and steel;
- perfect strain compatibility between steel and concrete at all reinforcement levels;
- no tension stiffening effects for the analysis of the cracked beams LT-0.8-Q and LT-0.5-Q (i.e. zero stress in the concrete of their tension zone).

These assumptions are common to nearly all models (6-13). The nonlinearity induced by cracking implies that iterations are necessary to satisfy simultaneously the equilibrium and compatibility equations at each time step.

The particular assumptions of the present model are :

- the integration procedure over the time steps. The integration method used in most of the models (6-13) is either the effective modulus method or the age adjusted effective modulus method (1). Both methods are suitable for problems involving a single load step (like prestressing and application of the dead load at the same age). Their application to multiple load steps problems is questionable. Our particular application involved at least three load steps. We therefore chose to apply a more general method, namely the numerical evaluation of the superposition principle by a trapezoidal rule (4);
- the interaction between the relaxation of the prestressing steel and the creep and shrinkage of concrete. In some of the models (6-13), the relaxation of the prestressing steel is ignored. Other models introduce a special "reduced" relaxation procedure which is particularly suited to the application of the age adjusted effective modulus method. The step-by-step procedure adopted in this analysis allows to handle the interaction rigorously by taking the stress in the prestressing steel at the beginning of each time step as fictitious initial prestress tension for deriving the intrinsic relaxation increment over the time step. The equations of the time-dependent development of the intrinsic relaxation are taken here from the book by Ghali and Favre (5) applied to the experimental values given in Table 3.
- the time-dependent model for predicting creep and shrinkage strains. The model uses the mathematical forms mentioned earlier (Eq.2 and Eq.3) but the parameters have been corrected as follows (Eq.4 and Eq.5) to introduce aging effects and to take into account the difference between the dimensions of the cross sections of the prisms and of the beams. These correction coefficients are similar to those used in the ACI 209 model (1).

$$\epsilon_{sh}(t) = -0.000480 \left( \frac{(t-t_{sh})}{78 + (t-t_{sh})} - \frac{(t_0-t_{sh})}{78 + (t_0-t_{sh})} \right) \quad (4)$$

$$J(t, t') = \frac{1}{26000 \text{ MPa}} \left( 1 + 2.1 \frac{1.365 t'^{-0.118} (t-t')^{0.6}}{15 + (t-t')^{0.6}} \right) \quad (5)$$

with  $t$ ,  $t'$ ,  $t_0$  and  $t_{sh}$  in days.  $t_{sh}$  is the age at the beginning of shrinkage and  $t_0$  is the age at the beginning of the analysis.

To the best of our knowledge, this paper reports for the first time the application of such kind of numerical analyses to actual experiments.

#### DISCUSSIONS OF RESULTS

The numerical model has been applied to all beams. To keep the illustrations readable and to limit their number, the computed values are reported here for PPC beams only in Fig.7, 8, 10, and 11. Although not perfect, the comparison seems satisfactory; better results could have been obtained by using a more sophisticated time-dependent prediction model for the evaluation of the shrinkage strain and the creep function. Some factors could anyway prevent from reaching a perfect agreement with experimental results :

- deduction of the curvature from strain readings is delicate, especially when the curvature is small or when the section is cracked;
- although great care has been taken from the experimental point of view, the exact magnitude and position of the prestressing force is not known exactly : small absolute errors in the position of the tendons lead to large relative errors in estimating the bending moment and the curvature at prestressing; for example, if the centroid of the prestressing reinforcement lies half a centimeter below its theoretical value, the actual bending moment at prestressing is underestimated by 8 %;
- the simple loading scheme faced here already leads to violation of the superposition principle of linear viscoelasticity. In fact, the superposition principle is known to be unable to cope precisely with unloading situations implying strain reversal. This situation is encountered here in all loaded beams, because the loading of the beams implies strain reversal in the strain history of all fibres. The magnitude of the error introduced by this commonly accepted approximation is unknown and probably small in comparison with other assumptions of the model like the overall characterization of creep and shrinkage by a single formulation for the whole cross section.

The typical experimental patterns in the curvature and mid-depth strain evolutions are perfectly predicted by the numerical analysis as appearing from Fig.7, 8, 10 and 11. This allows to use the numerical model to estimate the stress redistributions occurring between steel and concrete within the cross sections. Table 6 reports the computed stresses at prestressing and at five years at the lower fibre of the concrete section ( $f_c$ ) and at the lowest reinforcing ( $f_{s4}$ ) and prestressing ( $f_{p3}$ ) steel levels. If we limit our interpretation to the unloaded beams, it can be seen that the total prestress loss in the prestressing steel is larger for the PC beams than for the PPC beams (16.8 % for LT-1.0 (14) and 10.3 % for LT-0.5), but the relative loss of concrete

prestress increases with decreasing values of the degree of prestressing (32 % for LT-1.0 (14) and 65 % for LT-0.5). This more pronounced prestress loss in the concrete of PPC beams should be taken into account for the estimation of the long-term cracking moment. This has been recalled recently by Ghali (14) and shown numerically by Taerwe (15) for a section with similar characteristics. Bruggeling (16), however, believes that simplified methods are accurate enough to compute the prestress losses.

Another point worth of discussion is the deflection problem. It has been known for long that fully prestressed concrete beams may approximately be considered for many practical applications as homogeneous viscoelastic solids. Consequently, the time-dependent evolution of the deflection of uncracked PC beams is generally handled (with some provisions for the prestress losses) with the 1st principle of correspondence of linear viscoelasticity. If there is only one load step (prestressing and application of the dead load at the same age), the time-dependent camber follows an evolution which is somewhat proportional to the creep function. The experimental results represented in Fig.4 or in Fig.7, but also the experimental results reported by other investigators (17-19), show that this assumption is no longer verified for PPC beams. In fact, the quasi stabilization of the deflection (or curvature) for beam LT-0.8 and the change of sign for the deflection (or curvature) of beam LT-0.5 may hardly be represented by a time-dependent function proportional to the creep function. It has often been advocated that the time-dependent deflection of an uncracked PPC beam may be obtained by multiplying the deflection of the same beam without nontensioned reinforcement by a time-independent reduction factor accounting for the presence of the nontensioned reinforcement (e.g. factor  $\xi_r$ , Eq.3-9 in Ref.1). We believe that such computation methods do not recognize sufficiently the heterogeneous viscoelastic nature of uncracked PPC sections and the resulting time-dependent stress redistribution between concrete and reinforcing steel. An alternative viewpoint which has been proposed more recently (20) is the introduction of the influence of the nontensioned steel in a separate time-dependent deflection term accounting for all prestress losses. But this method still supposes that the initial prestressing force and the dead load act on a viscoelastic solid.

In connection with this time-dependent evolution, we must remark that, in some countries, structural engineers tend to design structural concrete by "balancing" the dead load with the prestressing reinforcement and let the rest of the design load be carried by ordinary reinforcing steel to meet the ultimate limit state requirements. We conclude from the test results and our interpretation of the stress redistribution that this procedure will not ensure automatically a zero deflection in the long-term, as would imply the classical load balancing method applied to the design of fully prestressed beams. This problem has been highlighted, for instance, by Favre and Perišić (21).

## CONCLUSIONS

This paper has reported experimental results from long-term sustained loading tests on fully prestressed (PC) and partially prestressed concrete (PPC) beams. The ultimate carrying capacity at five years was found statistically identical to the short-term carrying capacity. Maximum crack width at service load level never exceeded 0.2 mm, even under long-term loading.

The introduction of unsymmetrical ordinary reinforcement in an uncracked prestressed concrete section (case of uncracked PPC beams) produces non negligible viscoelastic heterogeneities and leads to a stress redistribution between concrete and steel. This appeared from experimental deflection or curvature time-dependent patterns and from a numerical analysis. This stress redistribution has to be taken into account to evaluate the serviceability limit states.

The deflection limit state seems to be a major consideration in designing prestressed structures. Partial prestressing, apart from other advantages which have not been recalled here (ductility, minimum passive reinforcement, ...) provides an effective mean of controlling the camber which becomes unacceptable in full prestressing if only a small part of the design load is permanent. On the other hand, the total long-term deflection of the partially prestressed beams with  $\lambda = 0.5$  is too large if the full design load is permanent. The degree of partial prestressing should therefore be chosen in function of the ratio of the part of permanent load to the design load. When used for the design of PPC structures, the application of the load balancing method requires also a careful serviceability check.

Analysis methods accounting rationally for the influence of nontensioned steel on the time-dependent behavior of prestressed concrete sections are available, but there is a need for serviceability oriented design methods which recognize adequately this influence.

## ACKNOWLEDGEMENTS

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## REFERENCES

1. ACI Committee 209, "Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures", (ACI 209R-82; reapproved 1986), American Concrete Institute, Detroit, 1982, 92 pp.
2. Espion, B. and Halleux, P., Long-Term Tests on Prestressed Concrete Beams (Essais de longue durée sur poutres en béton précontraint), Department of Civil Engineering Research Report, University of Brussels (U.L.B.), 1982 (in French).
3. Espion, B. and Halleux, P., Long-Term Tests on Partially Prestressed Concrete Beams (Essais de longue durée sur poutres en béton partiellement précontraint), Department of Civil Engineering Research Report, University of Brussels (U.L.B.), 1988 (in French).
4. Bažant, Z.P., "Numerical Determination of Long-Range Stress History from Strain History in Concrete, Materials and Structures (RILEM, Paris), V.5, N°27, 1972, pp.135-141.
5. Ghali, A. and Favre, R., Concrete Structures : Stresses and Deformations, Chapman & Hall, London and New York, 1986, pp.318-321.
6. Brøndum-Nielsen, T., "Stress Analysis of Concrete Section under Service Load", ACI Journal, Proceedings V.76, N°2, February 1979, pp.195-211.
7. Perišić, Z. and Alendar, V., "Effect of Nonprestressed Reinforcement on Prestress Losses and Serviceability Limit of Prestressed Members", International Symp. on Nonlinearity and Continuity in Prestressed Concrete, Univ. of Waterloo, Waterloo, Canada, July 4-6, 1983, V.1, pp.217-233.
8. Frey, J. and Trost, H., "Computation of Partially Prestressed Concrete Structures under Serviceability Conditions (Zur Berechnung von teilweise vorgespannten Betontragwerken im Gebrauchszustand)", Beton und Stahlbetonbau (Berlin), V.78, N°11, November 1983, pp.302-308, N°12, December 1983, pp.331-335 (in German).
9. Ghali, A., "A Unified Approach for Serviceability Design of Prestressed and Nonprestressed Reinforced Concrete Structures", Journal, Prestressed Concrete Institute, V.31, N°2, March-April 1986, pp.118-137.
10. Moustafa, S.E., "Nonlinear Analysis of Reinforced and Prestressed Concrete Members", Journal, Prestressed Concrete Institute, V.31, N°5, September-October 1986, pp.126-147.

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11. Inomata, S., "An Approach for Creep Analysis of Cracked Partially Prestressed Concrete Members", Journal, Prestressed Concrete Institute, V.32, N°1, January-February 1987, pp.104-125.
12. Al-Zaid, R.Z., Naaman, A.E. and Nowak, A.S., "Partially Prestressed Composite Beams under Sustained and Cyclic Loads", ASCE Journal of Structural Engineering, V.114, N°2, February 1988, pp.269-291.
13. Prasada Rao, A.S. and Jayaraman, R., "Creep and Shrinkage Analysis of Partially Prestressed Concrete Members", ASCE Journal of Structural Engineering, V.115, N°5, May 1989, pp.1169-1189.
14. Ghali, A., "Stress and Strain Analysis in Prestressed Concrete : A Critical Review", Journal, Prestressed Concrete Institute, V.34, N°6, November-December 1989, pp.80-97.
15. Taerwe, L.R., "Simplified Analysis of Stress Redistribution in Partially Prestressed Concrete Sections", ACI Structural Journal, V.87, N°1, January-February 1990, pp.93-98.
16. Bruggeling, A.G.S., "Partially Prestressed Concrete Structures : A Design Challenge", Journal, Prestressed Concrete Institute, V.30, N°2, March-April 1985, pp.140-171.
17. Hutton, S.G. and Loov, R.E., "Flexural Behavior of Prestressed, Partially Prestressed and Reinforced Concrete Beams", ACI Journal, Proceedings V.63, N°12, December 1966, pp.1401-1409.
18. Lambotte, H. and Thomas, P., "Partially Prestressed Concrete under Serviceability Conditions (Gedeeltelijk Voorgespannen Beton onder Gebruiksvoorwaarden)", Tijdschrift der Openbaren Werken van België (Brussels), N°3, 1973, pp.156-173 (in Dutch).
19. Lambotte, H. and Van Nieuwenburg, D., "Long-Term Behaviour of Partially Prestressed Beams", Partial Prestressing, From Theory to Practice (NATO ASI Series E : Applied Sciences, V.2), M.Z.Cohn, Editor, Martinus Nijhoff Publishers, Dordrecht, 1986, pp.109-123.
20. Tadros, M.K., Ghali, A. and Meyer, A.W., "Prestressed Loss and Deflection of Precast Concrete Members", Journal, Prestressed Concrete Institute, V.30, N°1, January-February 1985, pp.114-141.
21. Favre, R. and Perišić, Z., "Influence of a Large Amount of Asymmetrical Reinforcement on the Delayed Deformation of a Prestressed Concrete Structure (Influence d'une forte armature dissymétrique sur la déformation différée d'une structure précontrainte)", Ingénieurs et Architectes Suisses (Zürich), N°2, January 1983, pp.17-21 (in French).

**TABLE 1 -- DETAILING AND CHARACTERISTICS OF THE PRESTRESSING REINFORCEMENT ( $\lambda = 1.0$  : WIRE;  $\lambda = 0.8$  OR  $0.5$  : STRAND)**

| $\lambda$ | $A_{p1}$<br>(mm <sup>2</sup> ) | $d_{p1}$<br>(mm) | $A_{p2}$<br>(mm <sup>2</sup> ) | $d_{p2}$<br>(mm) | $A_{p3}$<br>(mm <sup>2</sup> ) | $d_{p3}$<br>(mm) | $f_{pu}$<br>(MPa) | $f_{py0.2}$<br>(MPa) | $f_{pi}$<br>(MPa) |
|-----------|--------------------------------|------------------|--------------------------------|------------------|--------------------------------|------------------|-------------------|----------------------|-------------------|
| 1.0       | -                              | -                | -                              | -                | 924                            | 285              | 1770              | 1585                 | 1200              |
| 0.8       | -                              | -                | 372                            | 260              | 372                            | 310              | 1892              | 1715                 | 1320              |
| 0.5       | 186                            | 260              | 93                             | 285              | 186                            | 310              | 1892              | 1715                 | 1320              |

**TABLE 2 -- DETAILING AND CHARACTERISTICS OF THE ORDINARY REINFORCEMENT (S 400)**

| $\lambda$ | $A_{s1}$<br>(mm <sup>2</sup> ) | $d_{s1}$<br>(mm) | $A_{s2}$<br>(mm <sup>2</sup> ) | $d_{s2}$<br>(mm) | $A_{s3}$<br>(mm <sup>2</sup> ) | $d_{s3}$<br>(mm) | $A_{s4}$<br>(mm <sup>2</sup> ) | $d_{s4}$<br>(mm) | $f_{sy(4)}$<br>(MPa) |
|-----------|--------------------------------|------------------|--------------------------------|------------------|--------------------------------|------------------|--------------------------------|------------------|----------------------|
| 1.0       | 201                            | 27               | 100                            | 100              | 100                            | 300              | 314                            | 370              | 435                  |
| 0.8       | 201                            | 27               | 100                            | 115              | 100                            | 285              | 565                            | 365              | 519                  |
| 0.5       | 201                            | 27               | 100                            | 115              | 100                            | 285              | 1272                           | 365              | 522                  |

**TABLE 3 -- INTRINSIC RELAXATION STRESS LOSS  $\Delta f$  IN % OF  $F_{p0}$  AT 1000 HOURS**

| under $f_{p0} =$ | 0.6 $f_{pu}$ | 0.7 $f_{pu}$ | 0.8 $f_{pu}$ |
|------------------|--------------|--------------|--------------|
| Wire 7 mm        | 1.31 %       | 1.85 %       | 5.10 %       |
| Strand 0.5"      | 1.48 %       | 1.55 %       | 1.78 %       |

**Note:**

The external diameter of the duct is 56 mm for the BBRV cables and 15 mm for the unbonded strands.

TABLE 4 -- DESIGNATION OF THE BEAMS AND LOADING PROGRAM

| Beam Designation | $\lambda$ | Age at pre-stressing (days) | Age at 1st loading (days) | Q at 1st loading (kN) | Age at 2nd loading (days) | Q at 2nd loading (kN) | Age at unloading (days) | Age at ultimate loading (days) |
|------------------|-----------|-----------------------------|---------------------------|-----------------------|---------------------------|-----------------------|-------------------------|--------------------------------|
| ST-1.0-(29)      | 1.0       | 14                          | -                         | -                     | -                         | -                     | -                       | 29                             |
| ST-1.0-(56)      | 1.0       | 14                          | -                         | -                     | -                         | -                     | -                       | 56                             |
| ST-0.8           | 0.8       | 14                          | -                         | -                     | -                         | -                     | -                       | 28                             |
| ST-0.5           | 0.5       | 14                          | -                         | -                     | -                         | -                     | -                       | 28                             |
| LT-1.0 (14)      | 1.0       | 14                          | -                         | -                     | -                         | -                     | -                       | 1797                           |
| LT-1.0 (45)      | 1.0       | 45                          | -                         | -                     | -                         | -                     | -                       | 1843                           |
| LT-0.8           | 0.8       | 14                          | -                         | -                     | -                         | -                     | -                       | 1884                           |
| LT-0.5           | 0.5       | 14                          | -                         | -                     | -                         | -                     | -                       | 1893                           |
| LT-1.0-Q/2 (56)  | 1.0       | 14                          | 56                        | 31.88                 | -                         | -                     | 1652                    | 1831                           |
| LT-1.0-Q (56)    | 1.0       | 14                          | 56                        | 63.75                 | -                         | -                     | 1645                    | 1851                           |
| LT-1.0-Q/2 (31)  | 1.0       | 14                          | 31                        | 31.88                 | -                         | -                     | 1648                    | 1839                           |
| LT-1.0-Q (31)    | 1.0       | 14                          | 31                        | 63.75                 | -                         | -                     | 1655                    | 1853                           |
| LT-0.8-Q         | 0.8       | 14                          | 28                        | 16.70                 | 85                        | 63.75                 | 1673                    | 1884                           |
| LT-0.5-Q         | 0.5       | 14                          | 28                        | 16.50                 | 84                        | 63.75                 | 1673                    | 1893                           |

TABLE 5 -- CONCRETE STRENGTHS AND CARRYING CAPACITY

| Beam            | $f'_c$ at<br>pre-<br>stressing<br>(MPa) | $f'_c$ at<br>ultimate<br>loading<br>(MPa) | $Q_u$<br>(kN) | $M_u$<br>(kN.m) | Actual<br>safety |
|-----------------|-----------------------------------------|-------------------------------------------|---------------|-----------------|------------------|
| ST-1.0-(29)     | 35.9                                    | 37.8                                      | 157.0         | 341.2           | 2.21             |
| ST-1.0-(56)     | 35.9                                    | 38.7                                      | 196.0         | 419.2           | 2.71             |
| LT-1.0 (14)     | 36.0                                    | 39.1                                      | 175.0         | 377.2           | 2.44             |
| LT-1.0 (45)     | 37.2                                    | 35.9                                      | 185.0         | 397.2           | 2.57             |
| LT-1.0-Q/2 (56) | 34.1                                    | 38.6                                      | 180.0         | 387.2           | 2.50             |
| LT-1.0-Q (56)   | 34.1                                    | 38.6                                      | 185.0         | 397.2           | 2.57             |
| LT-1.0-Q/2 (31) | 36.2                                    | 42.6                                      | 165.0         | 357.2           | 2.31             |
| LT-1.0-Q (31)   | 36.2                                    | 42.6                                      | 182.5         | 392.2           | 2.54             |
| ST-0.8          | 32.6                                    | 36.3                                      | 154.0         | 335.2           | 2.17             |
| LT-0.8          | 33.0                                    | 39.0                                      | 156.3         | 339.8           | 2.20             |
| LT-0.8-Q        | 31.9                                    | 36.7                                      | 152.0         | 331.2           | 2.14             |
| ST-0.5          | 32.4                                    | 34.9                                      | 164.0         | 355.2           | 2.30             |
| LT-0.5          | 33.4                                    | 38.3                                      | 167.5         | 362.2           | 2.34             |
| LT-0.5-Q        | 32.4                                    | 38.6                                      | 173.8         | 374.8           | 2.42             |

TABLE 6 -- COMPUTED STRESSES IMMEDIATELY  
AFTER PRESTRESSING AND AT 5 YEARS

| Beam            | $f_{p3,0}$<br>(MPa) | $f_{p3,t}$<br>(MPa) | $f_{s4,0}$<br>(MPa) | $f_{s4,t}$<br>(MPa) | $f_{c,0}$<br>(MPa) | $f_{c,t}$<br>(MPa) |
|-----------------|---------------------|---------------------|---------------------|---------------------|--------------------|--------------------|
| LT-1.0 (14)     | 1200                | 998                 | -92.9               | -281                | -15.0              | -10.2              |
| LT-1.0 (45)     | 1200                | 1026                | -92.9               | -250                | -15.0              | -10.9              |
| LT-1.0-Q/2 (56) | 1200                | 1040                | -92.9               | -191                | -15.0              | -4.8               |
| LT-1.0-Q (56)   | 1200                | 1080                | -92.9               | -102                | -15.0              | +0.6               |
| LT-1.0-Q/2 (31) | 1200                | 1041                | -92.9               | -187                | -15.0              | -4.9               |
| LT-1.0-Q (31)   | 1200                | 1082                | -92.9               | -95                 | -15.0              | +0.5               |
| LT-0.8          | 1320                | 1129                | -76.2               | -235                | -12.6              | -7.9               |
| LT-0.8-Q        | 1320                | 1243                | -76.2               | -63                 | -12.6              | cracked            |
| LT-0.5          | 1320                | 1184                | -38.8               | -131                | -6.2               | -2.2               |
| LT-0.5-Q        | 1320                | 1324                | -38.8               | +73                 | -6.2               | cracked            |

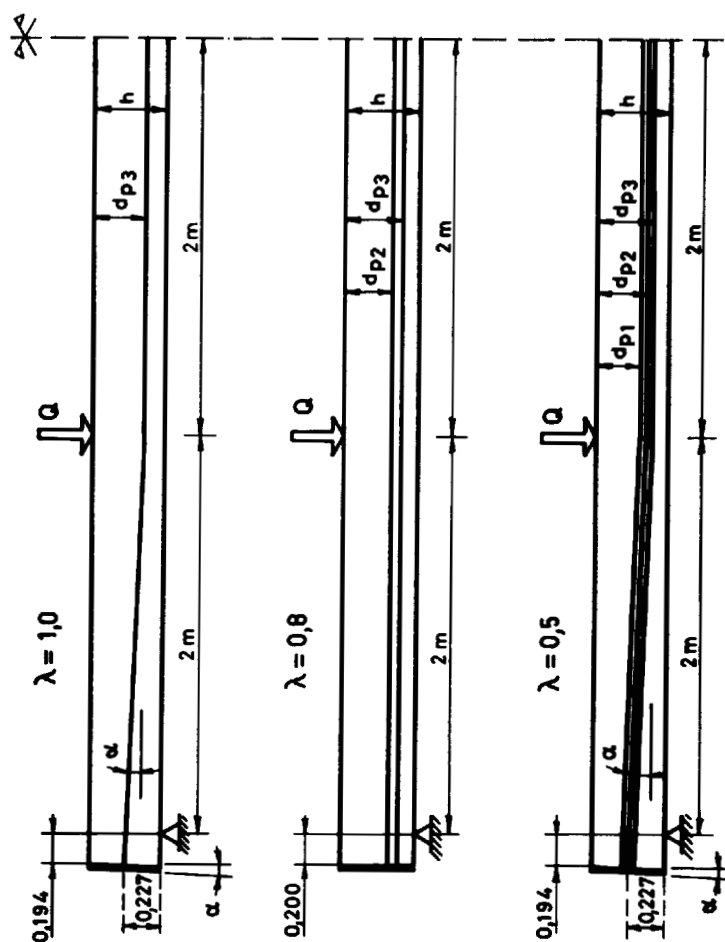


Fig. 1--Geometry of the prestressing tendon profiles

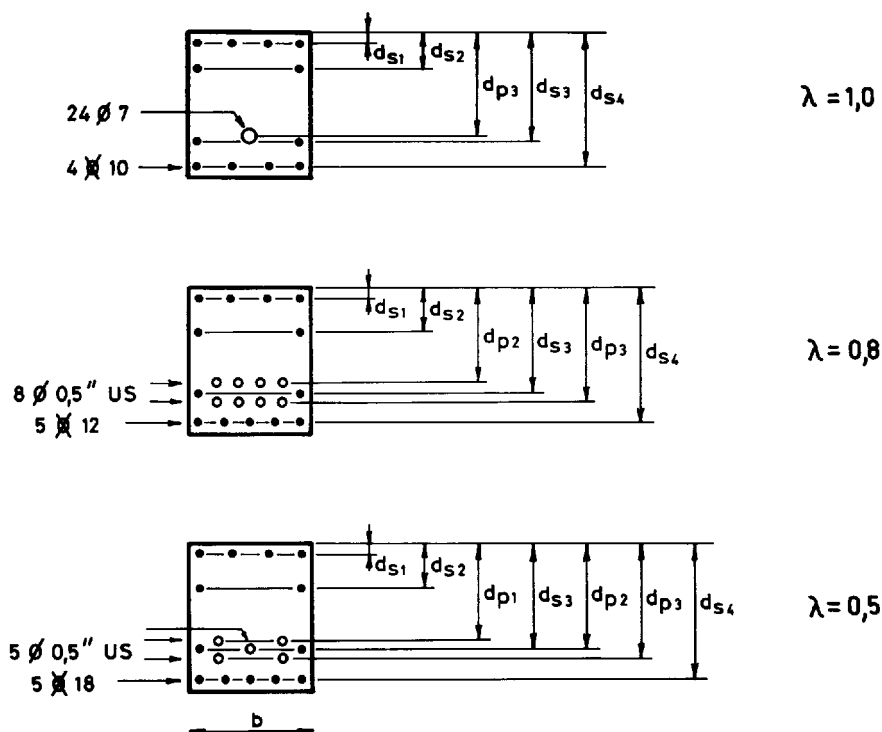


Fig. 2--Effective depths in the midspan sections

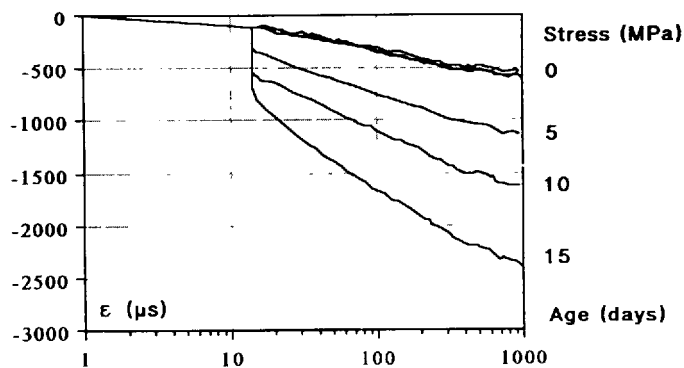


Fig. 3--Total mean strain of the creep and shrinkage prisms

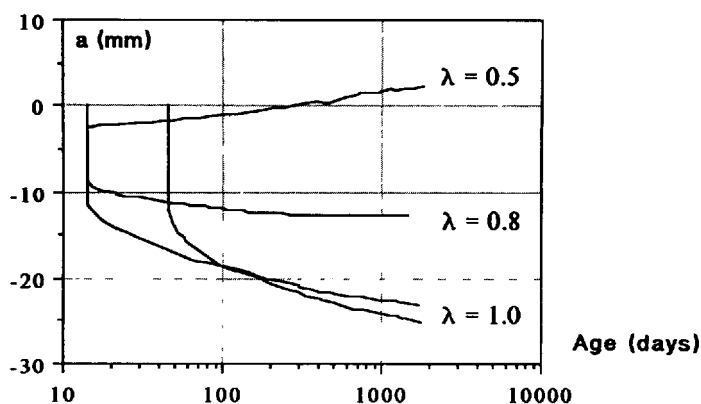


Fig. 4--Time-dependent deflection of the "unloaded" beams

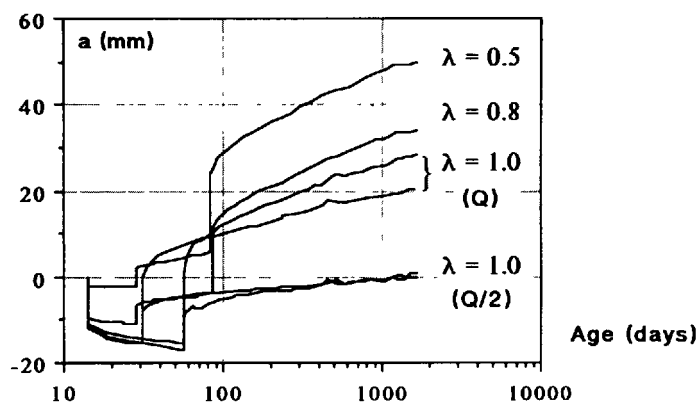


Fig. 5--Time-dependent deflection of the "loaded" beams

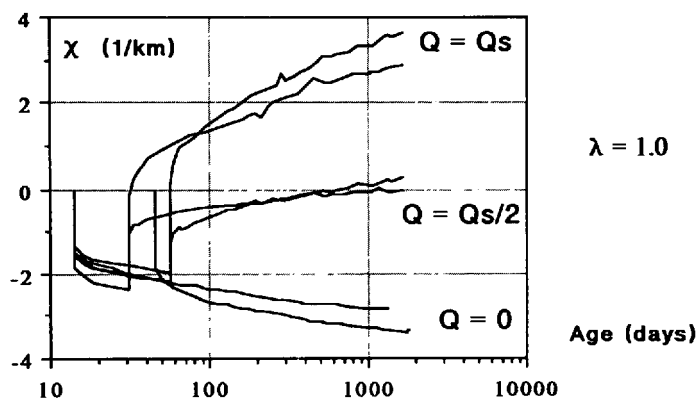


Fig. 6--PC beams, curvature of the midspan section

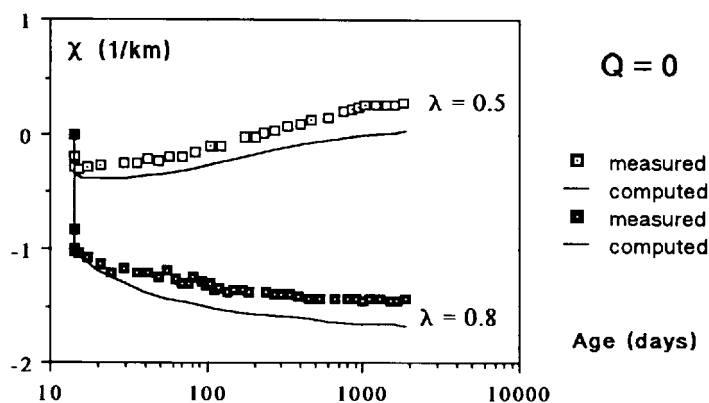


Fig. 7--PPC "unloaded" beams, curvature of the midspan section

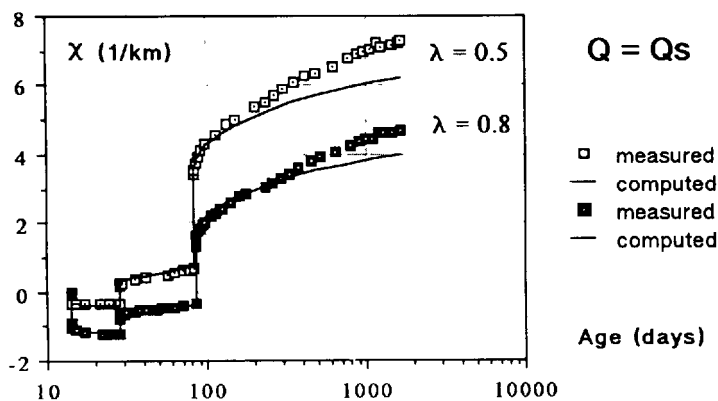


Fig. 8--PPC "loaded" beams, curvature of the midspan section

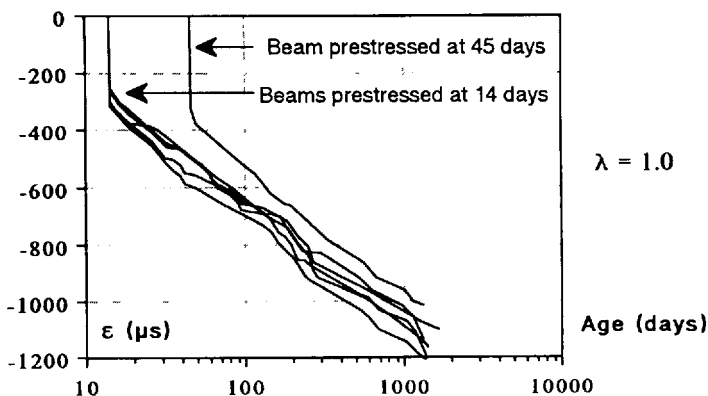


Fig. 9--PC beams, mid-depth strain of the midspan section

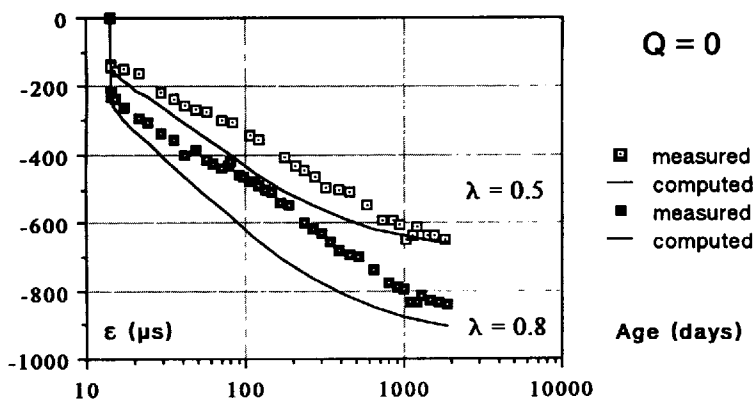


Fig. 10--PPC "unloaded" beams, mid-depth strain of the midspan section

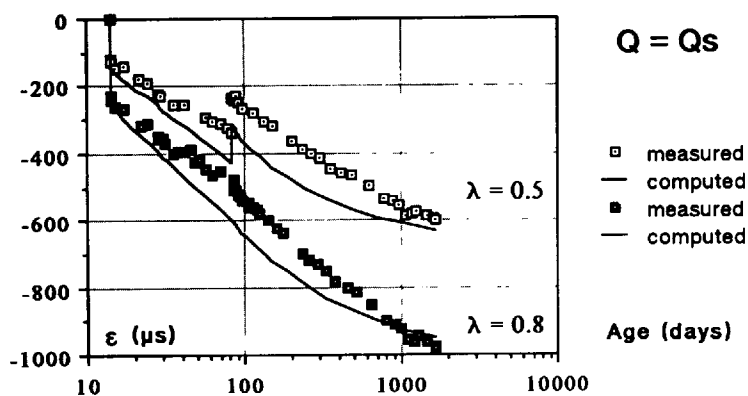


Fig. 11--PPC "loaded" beams, mid-depth strain of the midspan section

SP 129-3

# Creep Buckling of Uniaxially Loaded Reinforced Concrete Columns

by N. C. Mickleborough  
and R. I. Gilbert

**Synopsis:** The first stage of an experimental investigation into the instantaneous and time-dependent behavior of reinforced concrete columns under sustained load is reported. The experimental work described in the paper involves the testing of fifteen large scale columns in compression and uniaxial bending to obtain comprehensive creep deformation data for the prediction of long-term lateral deflection and instability. Information on the range of slenderness ratios and load levels which cause creep instability for rectangular symmetrically reinforced concrete columns is of particular interest. The experimental data is also for use in the development and calibration of a theoretical model for the prediction of creep deflection and buckling under sustained load.

The experimental set-up used in this investigation and described in the paper is designed for the simultaneous testing of five slender columns, each up to six metres in length. The loading frame is such that each column may have different length, different cross-sectional dimensions and reinforcement details, and be subjected to different combinations of axial force and bending moment. In the three series of tests presented here, column length, axial force and initial eccentricity are the major variables. The loading on each column is monitored independently and maintained automatically at a constant preset value throughout each test. A direct comparison of creep effects on column behavior is therefore possible as different loading parameters are varied.

**Keywords:** Buckling; columns (supports); creep properties; deflection; reinforced concrete; shrinkage; slenderness ratio; test equipment; tests.

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## INTRODUCTION

This paper presents the results of the first three series of laboratory experiments in an on-going research project into the time-dependent behavior of slender reinforced concrete columns. The project is divided into two parallel components; a large scale test program (of which this paper forms a part), and a theoretical study of the creep buckling problem. Reference 1 reports further on the latter aspect of the project.

The application of an eccentric compressive force to a slender column produces a strain/stress gradient across each cross-section and a consequential lateral deflection of the column. If the load is sustained over a period of time, creep of the concrete increases the strain gradient on each cross-section and therefore increases the lateral deflection with time. As the lateral deflection increases, the eccentricity of the applied load to the centroid of the displaced cross-section increases, and therefore the moment on the section also increases. This in turn causes additional curvature and lateral deflection. If the column is to remain stable at all times, the external moment must be balanced by the internal resisting moment on each cross-section. If the curvature on each cross-section of the column does not produce sufficient internal moment to balance the externally applied moment, the rate of increase of lateral deflection does not reduce with time and an instability failure occurs.

Slender columns are classified as members in which additional bending moments due to the slenderness effects are significant. Many factors effect the strength of a slender column, including the length, cross-sectional dimensions, material properties (including the creep characteristics), distribution of bending moment, magnitude of axial force, and the lateral restraint conditions for the member.

Slender columns must be designed for the additional bending moments due to slenderness effects. Provided the slenderness ratio  $kl_u/r$  is less than 100 (as specified by ACI 318-83 (2)) or 120 (as specified by AS3600-1988 (3)), the slenderness effects can be accounted for in design by use of the moment magnifier method of analysis. In this simplified method, the applied moment is increased by a moment magnification factor to account for the increased lateral deflection resulting from slenderness (the so-called P- $\Delta$  effect).

For members with  $kl_u/r$  greater than the values of 100 and 120, respectively, a rigorous method of analysis is required which accounts for the relevant material non-linearities (including creep, shrinkage and cracking), the geometric effects (where changes in deflection or displacement of the member influence the magnitude of internal actions), and the interaction of the member with the rest of the structure. The commentary to ACI 318-83 suggests that the limiting value of  $kl_u/r = 100$  represents the upper range of actual tests on slender compression members in frames. The limiting value of 120 in AS3600 is considerably less conservative than the ACI limit. In the initial drafts of this code, a limit of 140 was proposed.

The experimental program described in this paper is concerned with the behavior of slender columns with slenderness ratios above the limits specified in the two respective codes. Results from such experiments will prove useful in assessing the applicability of simplified design techniques for very slender columns. The data is also used for comparison purposes in the concurrent analytical investigation where both material and geometric non-linearities are taken into account in an iterative computer based solution procedure (1).

## EXPERIMENTAL PROGRAM

In this study, three groups of five slender columns have to date been manufactured and tested over periods up to 40 days. Each column was subjected to a constant sustained compressive load applied at a constant uniaxial eccentricity. The tests have been conducted in sets of five to achieve the advantage of eliminating as much variability between concrete mixes and environmental conditions as possible. The column cross-sectional dimensions and reinforcement details remained constant for all tests. In each set of five tests the magnitude of applied load and the eccentricity of loading were varied.

The first two sets of tests were conducted using six meter long columns. For the third set of tests the length of the columns was reduced to five meters. The cross-section dimensions and reinforcement details for all the columns are shown in Fig 1.

For each column, the measured concrete compressive strength (taken from standard 150 mm diameter cylinders), the length of the test specimen, the applied load and the initial eccentricity are given in Table 1.

The creep test apparatus is illustrated in Fig 2, and consists of five loading frames which permit the columns to be mounted and tested vertically. The loads are applied through the testing rig by hydraulic jacks mounted on the end of the lever giving a mechanical advantage of approximately 2. The steel loading arms are connected by a tendon through the ends of the arms and the hydraulic jack applies the load by tensioning the tendon. Pressure in these jacks is maintained by large capacity nitrogen/oil filled accumulators and the actual load applied to each column is monitored through load cells in series with and at the base of each of the columns. Electrical resistance strain gauges are used to monitor each load cell. Lateral deflection of each column is measured from dial gauges along the length of the column, namely at each end, the center and at quarter points.

Demec targets attached on the column surfaces in the plane of bending provided information on strain distribution across the respective sections. This was used for subsequent calculation of curvatures.

Fig 3 is a photograph of a column under test load with creep deformation occurring. The position of the dial gauges can be noted as can the lateral deflection due to creep effects. Fig 4 illustrates the lower loading arms with the hydraulic jacks. The load cells monitoring the load in the columns and the eccentricity of loading are also shown in this photograph.

Companion test cylinders were mounted in static creep test rigs to measure the variation of the creep coefficient with time for the concrete used in the columns. These creep specimens were standard 150 mm diameter cylinders with three sets of demec targets placed around the surface of each cylinder. Loads on the creep cylinders were maintained at values ranging from 0.2 to 0.4  $f_c$ . Shrinkage specimens were also used to provide shrinkage data for the tests.

**Table 1** Details of each test column.

| Series | Column No. | Column Length (mm) | $f_c$ (MPa) | Eccentricity (mm) | Load (kN) |
|--------|------------|--------------------|-------------|-------------------|-----------|
| A      | 1A         | 6000               | 32          | 60                | 45        |
|        | 2A         | 6000               |             | 45                | 75        |
|        | 3A         | 6000               |             | 30                | 73        |
|        | 4A         | 6000               |             | 15                | 73        |
|        | 5A         | 6000               |             | 0                 | 90        |
| B      | 1B         | 6000               | 27          | 60                | 45        |
|        | 2B         | 6000               |             | 45                | 43        |
|        | 3B         | 6000               |             | 45                | 60        |
|        | 4B         | 6000               |             | 30                | 58        |
|        | 5B         | 6000               |             | 15                | 54        |
| C      | 1C         | 5000               | 39          | 60                | 55        |
|        | 2C         | 5000               |             | 45                | 60        |
|        | 3C         | 5000               |             | 45                | 65        |
|        | 4C         | 5000               |             | 30                | 65        |
|        | 5C         | 5000               |             | 15                | 70        |

## EXPERIMENTAL RESULTS

Representative experimental results of the three test series are presented in graphical and tabular form.

Throughout the test period, none of the slender columns failed due to creep buckling despite the large slenderness ratios. However, substantial time-dependent midlength deflections were recorded especially for the 6 meter long