DECLARATION

The writer hereby declares the work contained in this thesis with the exception of quoted sources, is his own work and has not been submitted to any other university.

G.E. Hoppe

GERT E. HOPPE

The writer has successfully completed course work to the value of 30 credits towards the M.Sc degree. This thesis therefore represents a quarter of the credit requirements for the degree.
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1. SYNOPSIS

The object of this thesis is

"To investigate the actual creep, shrinkage and elastic strains in a prestressed concrete bridge and to compare these with the results of creep and shrinkage tests carried out in the laboratory in accordance with ASTM C512 Specification on concrete cylinders made using the same aggregates and mix proportions as used for the prestressed concrete bridge and also with the theoretical values calculated using the CEB-FIP recommendations."

A literature survey has been carried out to determine the present state of knowledge on the creep, shrinkage and elastic shortening of concrete structures and how this knowledge is used in design.

Measurements of actual creep, shrinkage and elastic strain in concrete of the Brakwater South bridge near Windhoek have been recorded since its construction in 1978.

At the same time in the laboratory concrete test cylinders and cubes were prepared using the same aggregates and mix proportions as used for the construction of the bridge. Some of the specimens were cured and tested in accordance with BS 1881 to determine the strength of the concrete and the Modulus of Elasticity. Other specimens were cured and stored at controlled temperature and humidity conditions in accordance with ASTM C512-76 to determine the creep and shrinkage characteristics of the concrete under controlled laboratory conditions.

Using the latest CEB-FIP and BS 5400 design codes the theoretical design values of creep, shrinkage and elastic shortening are calculated, based on actual temperature, humidity conditions prevailing at the site since the construction of the bridge.

The values of creep, shrinkage and elastic shortening actually measured on site are compared with the theoretical design values based on small scale laboratory tests and tentative design recommendations applicable to local conditions are produced.
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3. INTRODUCTION

3.1 GENERAL

Concrete exhibits these important characteristics:

(a) Like many other structural materials, concrete is to a certain degree elastic.
(b) When kept under sustained load, concrete deforms slowly with time, the movement being commonly known as creep.
(c) Whether subjected to load or not, concrete contracts on drying, the shortening being known as shrinkage.

Engineers have since the turn of this century been aware of these characteristics of concrete, but it is only comparatively recently, in connection with the loss of prestressing found in prestressed concrete and the increase in deflections with time of large span beams that it has become necessary to determine the magnitude of the creep, shrinkage and elastic shortening and deflection of concrete.
Concrete structures can only be designed successfully if the characteristics of concrete behaviour such as creep, shrinkage and elastic strains are known. With the tremendous upsurge in the construction of prestressed concrete bridges in the early fifties it was realized that it was essential that the actual behaviour of bridges had to be investigated in order to assess the accuracy of the assumed design criteria which were mostly based on the theoretical work of Dr Dischinger.

One of the first actual measurements of creep and shrinkage on actual prestressed bridges were carried out by Dr U. Finsterwalder (12.7)* in Germany which confirmed that there was reasonable agreement between the predicted and actually measured movements and deflections.

In Britain, a programme of investigation into the creep and shrinkage of bridge structures was begun in 1962 and reported on by R.G. Tyler (12.1)* who found that the CEB-FIP recommendations gave values nearest to those measured if allowance was made for the fact that the relative humidity in Great Britain was higher than on the Continent.

While similar research has since then been carried out in other countries, very little work in this field has been done in Southern Africa.
Experimental Work in Southern Africa

In the sixties sophisticated prestressed concrete structures were designed and constructed in Port Elizabeth in accordance with the latest British Code - CP 115 (12.52)* available. After a few years it was noticed that these structures were not behaving as expected, the joint and bearing movements were much greater than predicted, indicating that creep and shrinkage properties of the local concrete was not the same as that of the overseas concrete on which the design code factors were based.

In order to investigate the problem, a series of creep and shrinkage tests were conducted by B.J. Mackenzie (12.6)* in the laboratory in accordance with ASTM C512-66T (12.48)* on specimens made from Port Elizabeth aggregates and cement. The findings of these tests showed clearly that creep strain of the concrete was considerably higher than the figure recommended in CP115 (12.52)* which explained why the observed movements of the bridges were greater than predicted. Since then many more laboratory creep and shrinkage tests have been conducted on concrete made with aggregates from different sources.

As a result of these findings, it has become the accepted practise for the design of prestressed bridges to use the basic values of creep, shrinkage and elastic strains obtained from laboratory tests in accordance with ASTM C512-66T (12.48)* on concrete specimens made with the local aggregates and cement and to adjust these values for actual site conditions of temperature, humidity and size effect in accordance with CEB-FIP (12.44)* code.

To date very little correlation has been done to determine the accuracy with which such laboratory tests under controlled conditions actually represent, the true, complex behaviours of concrete structures in-situ. Beside this current research project, the National Building Research Institute of the CSIR (12.53)* are conducting independently a long term investigation of the deflections, creep and shrinkage of a number of bridges in South West Africa.
4. STATE OF THEORY

4.1 BASIC THEORY

4.1.1 Literature Survey

The number of Papers dealing with creep and shrinkage that have been published in the last two decades have increased enormously. Considerable research and testing have been and are still being undertaken on the subject of creep throughout the world. This work ranges from using the electron-microscope to postulate the cause of creep in the microstructure of the creep-inducing material of concrete, namely the hardened cement gel, to actual observations of structures.

Unfortunately for the practising Engineer, most of the available information is fragmented and in addition there are many areas where no agreement has yet been reached by specialists in this field (Bazant - 12.10). This is most disconcerting, since most practising Engineers in design offices do not have the time to make their own comparisons with test data and with the various methods of analysis available. They need standard recommendations, agreed to by the creep specialists in committees, which they can take for granted.

4.1.2 Early-Age Movements

(i) Heat of Hydration of Cement

The hydration of cement is an exothermic reaction and the amount of heat generated per unit mass of cement at any specified stage of hydration is known as the heat of hydration of cement and varies
with the water : cement ratio and with the temperature of curing.

(ii) Laboratory Tests

An expansion normally occurs on setting, the amount measured depending upon the stiffness of the type of gauge buried in the concrete. During the initial temperature rise, it is generally accepted that the coefficient of expansion of the wet mix is greater than that of the wire of the gauge owing to the presence of free water. In consequence the gauge records the excess expansion of the mix over that of the gauge, plus superimposed strains arising from the stresses induced. The stiffness of the gauge tube also influences the reading. Stress values would be difficult to assess during this phase as the concrete gradually sets, to harden on or just before the peak hydration temperature. Plastic flow is possible to equalise differential expansion, & at this stage "plastic cracking" in tension zones can occur. (Tyler - 12.11)*

The early-age movement and temperature in an insulated concrete block in the laboratory is shown in Fig. 4.1.1

* Movement and temperature in insulated concrete block in the laboratory.

FIGURE 4.1.1 (TYLER- 12.1)
Up to 20 hours, when the peak of the hydration temperature occurs, the gauges recorded expansion and from 20 to 120 hours a contraction which was mainly a thermal drop caused by different coefficients of expansion for the wire of the gauge and the concrete. Then from 120 to 675 hours true shrinkage at constant temperature caused by moisture loss occurred. Measurements taken on casting suggest that shrinkage due to moisture was small in well cured concrete soon after casting.

(iii) Actual Structure Tests

On full scale structures because of the size of cross sections, large temperature differences occur at a cross section compared to the small laboratory test blocks where the temperature was constant throughout. This fact is well illustrated by the results obtained from the Chiswick cantilever (Tyler - 12.11)* as shown on Fig 4.1.2

The results below show that at the time of the peak hydration temperature 27 hours after casting, the concrete temperature at the inside of this section was about 20°C higher than at the outside. Thus, as the inside of the section was hotter than the outside during the first day after casting, the inside would tend to expand more than the outside due to temperature rise, and during cooling on the following days would tend to contract more. This would give rise to different strains within the cross section. During the first day a compressive strain would be expected to develop on the inside of the section and a tensile strain on the outside, whereas on cooling a reverse trend would apply, but under changed conditions as by then the concrete would be hard. It is therefore, likely that there is a residual tensile strain at the centre of the cross section.

An important point to note is that based on the above and other research such as the measurement of long term strains in the Medway Bridge (Tyler 12.12)* it is clear that the effects of the early-age strains caused by the heat of hydration of the cement are the most usual cause of cracking which occurs soon after casting in well cured concrete.

STATE OF THEORY
(iv) **Conclusion**

Based on the finding of the researchers, the site shrinkage measurements for this investigation have been commenced after the heat of hydration has been dissipated, in order that the difficulty of separating it from contraction due to thermal drop is avoided.

In English climate the heat of hydration has dissipated after about 120 hours which then also corresponds to the stripping of shutters. In South Africa shutters can be stripped earlier, and for this investigation the shrinkage measurements were commenced 72 hours after casting of the concrete when it had cooled down.
4.1.3 Elastic Strains

(i) Modulus of Elasticity

When a load is applied to a structural material it deforms. If on removal of the load the recovery is both complete and immediate the material is considered to be perfectly elastic, which concrete is not.

If the ratio of the applied compressive stress to the longitudinal strain produced is constant, the constant is called the "Modulus of Elasticity". The stress-strain relationship for concrete is not constant or linear, mainly due to creep or plastic deformation, particularly at slow rates of loading. A portion of the curve may however, be regarded as effectively linear and at stresses within this range the elastic modulus may be taken as the slope of this linear portion and it is referred to as the "initial tangent modulus". If the stress is above that at which the stress-strain relationship deviates from linearity, two further forms of elastic modulus may be considered, namely the "tangent modulus" as represented by the slope of the tangent to the curve at the particular stress, and the "secant modulus" represented by the slope of the line connecting the origin to the point of the curve corresponding to the stress selected as shown on Fig 4.1.3

Factors Affecting Modulus of Elasticity

There are a number of factors that affect the modulus of elasticity of concrete such as the mix proportions, the type of aggregate, the
the shape of the aggregate, temperature, the age of the concrete whether it is wet or dry at the time of test, and incomplete consolidation.

**FIGURE 4.1.3** (FULTON 12.57)*

Relationship between Elastic Modulus and Strength

The factors which affect the modulus of elasticity of concrete do not however, always have a correspondingly similar effect on concrete strength. The use of an aggregate with a higher modulus of elasticity does not necessarily produce a concrete of greater strength although it will increase the modulus of the concrete. In view of this it has not been possible to establish a unique relationship between the modulus of elasticity and its compressive (or flexural) strength. Limited correlations have nevertheless been obtained and according to CP 110 the elastic modulus of concrete usually lie within the ranges given in Table 4.1.4.

<table>
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<th>Compressive strength (cube) MPa</th>
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<td>Mean value</td>
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**TABLE 4.1.4** (FULTON 12.57)*

STATE OF THEORY
The results of investigations carried out on concretes made with South African aggregates and covering a wide range of strength are shown on Fig 4.1.5

![Graph showing the relationship between static modulus and compressive strength of concrete.](image)

The values refer to the static modulus and the cube strength of the concrete and may be compared with the values given by CP 110. The mean value corresponds to the formula

\[ E = 4.9 f_c^{1/3} \]

where \( E \) is in GPa and \( f_c \) is the compressive strength in MPa.

(ii) **Laboratory Determination of "E"**

A standard procedure for determining the static modulus of elasticity in compression is given in BS 1881 - Part 5 (12.50)*

The concrete test cylinders, which are 150 mm in diameter and 300 mm long are repeatedly loaded in compression to about \( \frac{1}{3} \) of their ultimate compressive and the longitudinal strain measured with a suitable extensometer.

In addition the Modulus elasticity can be determined as the first...
stage of the laboratory creep test, although in this case the loading is not cycled as required by BS 1881 - Part 5.

(iii) Actual Structure Tests

The prestressing of actual structures normally takes hours or days to complete, and the strain measurements on the full scale thus include some creep as well as elastic strains.

(iv) The Effect of Site Conditions on "E"

The effect of site conditions upon the size in the value of modulus of elasticity was shown by specimens cast of the new London Bridge (Tyler 12.1)*. Six specimens were cast with a precast unit and kept outside with the unit for the first 24 hours. Thereafter four specimens were brought inside, two of which were placed in water at 20°C and two in air at 20°C at 80% RH, the third pair was kept outside sheltered from the rain. The modulus of elasticity results are given in Fig 4.1.6 for a year following casting.

![Graph showing modulus of elasticity for samples of concrete from London Bridge (cast 1 March 1969).](image)

Values of modulus of elasticity for samples of concrete from London Bridge (cast 1 March 1969).

FIGURE 4.1.6

(TYLER 12.1)*

It can be seen that the specimens stored outside showed a slower gain in modulus of elasticity than those inside and the researcher concluded that specimens stored outside should be used in order to determine the modulus of elasticity of a bridge. However, an outside exposure hut is required to give roughly similar conditions to the outside specimens as those prevailing at the bridge site and
the specimens must be waterproofed to give the same surface/volume ratio as the bridge, in order to try to eliminate the size effect as far as possible.

(v) Conclusions

For purpose of this investigation which is to determine creep and shrinkage and elastic strain of concrete, the modulus of elasticity of the actual bridge and laboratory concrete specimens was only required to be known at the time of applying the one-stage prestress at 14 days after casting and also at 28 days after casting. The Modulus of Elasticity of the actual bridge at the time of stressing was obtained from the observed strains and the Modulus of Elasticity of laboratory specimens in accordance with BS 1881.

However, had it been necessary to calculate stresses in the bridge from measured strains under long term loading then the actual effective modulus of elasticity which is the ratio of stress to total movement comprising elastic strain, creep and shrinkage up to the time considered would have been required. The effective modulus of elasticity of concrete has been found to be considerably lower than the static modulus of concrete (Tyler 12.15)*

4.1.4 Creep

(i) Definition

Creep is defined as that portion of the time dependent strain which is induced by stress.

Under normal conditions of loading the instantaneous strain recorded depends on the speed of application of the load and includes thus not only the elastic strain but also some creep; but this is not of practical importance as it is the total strain induced by
the application of load that really matters. Since the modulus of elasticity of concrete increases with age, the elastic deformation gradually decreases and strictly speaking creep should be taken as strain in excess of the elastic strain at the time at which creep is being determined Fig 4.1.7

\[ \begin{align*}
\text{Elastic strain } & \text{Time since Application of Load} \\
\text{Creep} & \text{Shrinkage} \\
\text{Total deformation} & \\
\end{align*} \]

**Figure 4.1.7 (NEVILLE 12.43)**

Often, the modulus of elasticity is not determined at different ages and creep is simply taken as an increase in strain above the initial elastic strain. This alternative definition, although theoretically less correct, does not introduce a serious error and is often more convenient to use (Neville 12.43).

(ii) **Concrete as a Material**

Concrete can be regarded as a two-phase material consisting of a mixture of naturally occurring gravels or crushed rock, bonded together by a hydrate formed during the reaction of the cement with the mixing water (Fig 4.1.8). The aggregate, which is normally graded in size from 38 mm down to 100 µm can occupy up to 75% of the concrete by volume. Figure 4.1.9 illustrates the effect upon creep of variation in aggregate volume occurring in the normal range of concrete mixes. Changing from a 1:1:2 to a 1:2:4 mix only increases the aggregate volume from 60% to 75%, yet this causes a reduction in creep by as much as 50%.

The majority of aggregate in common use in South Africa can be considered as inert with respect to creep and, as such, its presence in concrete reduces creep in essentially two ways:
(1) The relative volume of the creeping material, the hardened cement paste, is reduced by the presence of the aggregate.

(2) The aggregate has a stiffness between 10 to 20 times greater than that of the cement paste and thereby restrains the high potential creep likely within this component. The effect of aggregate stiffness upon concrete creep is illustrated in Fig 4.1.10.

![Fig 4.1.8: The Relative Volume of components in concrete (CONCRETE SOCIETY 12.3)*](image)

![Fig 4.1.9: The influence of aggregate content upon concrete creep (CONCRETE SOCIETY 12.3)*](image)

![Fig 4.1.10: The effect of aggregate modulus of elasticity upon creep of concrete(17). Creep assumed as unity for $E_{agg} = 6.9 \times 10^4 \, \text{N/mm}^2$. Coarse aggregate content = 45 to 55% by volume. Relative creep = $\frac{\text{creep at } E_{agg}}{\text{creep at } 6.9 \times 10^4 \, \text{N/mm}^2}$](image)

Various theories associated with the state of moisture in the gel structure exist, attempting to explain why, under sustained load, hardened cement paste creeps and it seems likely that two or more different mechanisms are involved (Mackenzie 12.58)
With reference to the above diagram creep behaviours can be explained in terms of a combination of seepage of adsorbed water and viscous flow at lower stresses and intercrystalline slip at higher stresses. Under stress some of the adsorbed water (a very thin layer of water molecules adhering to the surface of gel particles) is released and seeps into empty pores. Near the surface some of this water evaporates if it can. This process is largely reversible except where the gel structure has changed or grown in the meanwhile. This seepage probably accounts for the relatively rapid initial creep and creep recovery. At the points of contact of gel particles covered with adsorbed water further movement can take place by much slower and irreversible viscous flow. Similar viscous flow probably also occurs along the interface of gel and aggregate particles.

In the normal working range, creep is proportional to the applied stress. At higher stresses additional creep strain occurs and this is probably due to intercrystalline slip and internal rupture of bond. This too is largely irreversible.

(iii) Factors Influencing Creep

In most investigations, creep has been studied empirically in order to determine how it is affected by various properties of concrete. A difficulty in interpreting many of the available data arises from the fact that in proportioning concrete it is not possible to change one factor without at least altering one other.
The factors influencing creep of concrete can be conveniently divided into two main categories: (Concrete Society 12.3)*

(Mackenzie 12.58)*

(a) Intrinsic Factors - these relate to the internal state of the material

(b) Extrinsic Factors - factors such as external environment and member size which influence the internal state of the material

(c) Secondary Effects - these factors have only secondary effect under "normal" circumstances.

(a) Intrinsic Factors

Structure of Cement Paste:

The development of the gel structure of the hydrated cement has significant effects upon creep. Thus if the product of hydration "the gel" is dense, then the creep will be low.

The density of the gel is partially controlled by the initial spacing of the cement grains and close packing of the cement grains is brought about by a low water/cement ratio and good compaction.

The general effect of water/cement ratio upon creep is shown in Fig 4.1.12

![Graph showing the influence of water-cement ratio on ultimate specific creep in Hummel's tests.](image-url)
Proportions and Stiffness of Aggregate

The aggregate in concrete is normally inert in that it does not creep under load. Its presence in the cement paste has a modifying influence upon the creep in that it normally acts as a restraint against the movement of the paste. The degree of restraint depends upon the proportion and upon the stiffness of the aggregate and is obviously related to the modulus of elasticity of the aggregate. See Figures 4.1.13 and 4.1.14

![Graph showing creep of concrete made with different aggregates](image)

**FIGURE 4.1.13 (NEVILLE 12.64)*

(b) Extrinsic Factors

Temperature:

In the majority of structures, especially bridges, the range of bulk operating temperatures is small and this factor is relatively unimportant. Figure 4.1.15 shows the influence of loading age and the effect of temperature between 20°C and 95°C upon the creep of moisture-stable mass concrete.

However, where high temperatures occur such as in a prestressed concrete nuclear pressure vessel, creep strain is considerably higher and the effect is important.
Relation between ultimate specific creep of concrete and modulus of elasticity of the aggregate used.

FIGURE 4.1.14

The effect of temperature upon the creep of moisture-stable mass concrete. All concretes loaded at 13.8 N/mm². Time under load extrapolated to 30 years (T.W.C. data).

FIGURE 4.1.15

(CONCRETE SOCIETY 12.3)*
Relative Humidity

The relative humidity of the air is a most important factor affecting the creep of concrete, especially in the drier inland areas of Southern Africa. For a given concrete loaded in a moist state, the creep is higher, the lower the relative humidity as shown in Figure 4.1.16

![Figure 4.1.16](image)

Creep of concrete cured in fog for 28 days, then loaded and stored at different relative humidities

It can be seen from Figure 4.1.16 that it is during the early period under load, when the concrete is drying, that the effect of low RH is most severe. It is important to note that it is the average daily, monthly etc. relative humidity that must be allowed for, (see Appendix 11.4) rather than the extreme daily, monthly etc. values.

Size of Member

The size of a concrete member determines the degree to which changes in ambient temperature and relative humidity will affect the concrete behaviour. Obviously small members are influenced by diurnal changes in temperature and relative humidity to a far greater extent than massive members, see Figure 4.1.19.
The volume/surface ratio does not reflect perfectly variations of both size and shape. However, the degree of correlation found between both shrinkage and creep, and volume/surface, is satisfactory for purposes of practical design (Hansen and Mattock 12.40)*.

**Applied Stress**

Concrete subjected to sustained loads of about 75% to 80% of its short term ultimate strength will eventually fail due to the formation of cracks in the cement paste and at the aggregate paste interface.

Up to about 40% of the concrete strength, creep increases with stress in an approximately linear manner for loaded concrete in both moisture-stable and drying conditions.
Secondary Effects

Type of Cement:
Unless stated otherwise all research data refers to ordinary Portland Cement. For a given stress at a given age, it may be assumed that creep increases with the type of cement as follows:
- aluminous, rapid-hardening, ordinary Portland, Portland blast furnace, low-heat, Portland/possolan (maximum creep).

Compaction

In normal circumstances, the concrete is fully compacted and unsegregated. If it is not, the voids remaining will affect creep in precisely the same manner as the capillary spaces formed in the normal course of hardening.

Grading, size and shape of the Aggregate

Experiments have shown that the effects are indirect only in that workability is greatly changed by an alteration in grading. This results in a different choice of mix ratios and the influence upon creep must be found from the relative volumes of the hardened cement and aggregate components rather than from the grading of the aggregate itself.

Admixtures

Research relating to the effect of admixtures on creep deformation of concrete is not extensive but much of it indicates that many commonly used admixtures exert a detrimental effect on the creep characteristics of concrete (Fulton 12.57)*

However, most admixtures are introduced to improve workability or the rate of hardening of the concrete. It has been found that where mixes with admixtures are designed to have the same strength and slump as mixes without admixtures then the reduction in water content (and corresponding creep) achieved by the use of admixtures compensates for the additional creep that results from...
the use of admixtures (Peters 12.59)*

Nevertheless, for situations in which the effects of creep are critical, the use of admixtures in concrete should be preceded by laboratory tests.

4.1.5 Shrinkage

(i) Definition

Shrinkage is defined as that portion of the time-dependent strain which would occur without the imposition of stress from external loads.

The various volume changes in concrete can be classified as follows:

(a) Changes due to Cement Solution, Water Absorption

When water is first added to cement a certain amount of the latter goes into solution with a consequent reduction in volume of the system. With porous aggregates, some water is absorbed by the aggregate resulting in a further reduction in volume.

(b) Changes due to Bleeding

The sedimentation of the solid particles in fresh concrete commonly results in the appearance of clear water on its surface. When this water is removed or is allowed to evaporate the volume of the hardened concrete is obviously less than the volume of the original system by an equivalent amount.

(c) Autogenous Volume Changes of Concrete

The system cement plus water contracts as hydration proceeds owing to the formation of products which have lesser volumes.
than the sum of the volumes of the original components taken separately. Although the volume of the total system may decrease, the absorbed volume of solids increases, since the newly-formed solids are the result of the chemical combination of the original solids with water.

While autogenous volume change will always result in a reduction in the absolute volume of paste, the actual change in overall volume may be either an expansion or a contraction, depending on the availability of free water in the pores.

(d) **Heat of Hydration Movements**

Due to the heat of hydration of the cement, the temperature of a mass of fresh concrete may rise considerably. The thermal coefficient of expansion of wet concrete can be assumed to be equal to or greater than that of hardened concrete and if the concrete is insulated to prevent loss of heat, appreciable expansion may result.

(e) **Drying Shrinkage**

When water saturated concrete is exposed to a dry atmosphere (e.g., when formwork is removed from concrete) the exposed surface of the concrete immediately starts to lose water. The rate of evaporation initially depends upon relative humidity, temperature, water/cement ratio and the exposed surface area of the concrete.

Water is lost by a capillary process, water in the cement paste being removed progressively by evaporation from the menisci in the larger capillary pores. The capillary tension that develops in the residual water induces compressive stresses in the concrete and as a result the concrete shrinks.

As drying continues the menisci recede away from the concrete surface and the water held by physical forces to the internal surfaces is lost. Removal of this absorbed water changes the surface energy of the solid phase and causes further shrinkage.

Loosely bound water of hydration may also be lost giving rise to additional shrinkage.
(f) **Carbonation Shrinkage**

Carbonation is caused by carbon dioxide (CO₂) from the atmosphere diffusing slowly into concrete and reacting with cement hydrates to form calcium carbonate. Shrinkage is increased by this process because particles which are supporting drying stresses are being chemically eroded. Carbon dioxide penetrates only a centimetre or so into the surface of concrete and so carbonation shrinkage is only significant in thin concrete elements.

(g) **Ageing**

In the case of large concrete members which dry slowly a further factor, namely ageing, may significantly affect concrete shrinkage. Prolonged curing can produce changes in the cement hydrates which reduce shrinkage (Parrot 12.34)*

(ii) **Relative Importance of Different Types of Shrinkage**

The different types of shrinkage described above have varying importance in their final effect on the concrete.

Volume changes occurring while the concrete is still sufficiently plastic to accommodate itself to such changes have little effect on the final concrete. Into this category fall (a) changes due to cement solution, water absorption (b) changes due to bleeding and (c) the early portion of autogenous volume changes of concrete.

Of greater importance are the volume changes that occur once the concrete mass has established its final structures. The stage at which this unaccommodative internal structure is reached is not known for certain but it would appear to be attained more than 12 hours after water is first added. It is around this stage that (d) the maximum heat of hydration is reached and on cooling thermal shrinkage will result (See Section 4.1.2 of this report). The autogenous volume changes (c) occurring after permanent set has been established are small and can be neglected as well as (f) the carbonation shrinkage except for very thin members.

It is therefore evident that drying shrinkage (which does not include the thermal contraction on cooling after a heat of cement
hydration temperature rise) is the most important factor.

(iii) Factors Influencing "Drying" Shrinkage

The testing of concrete and mortars for shrinkage using standard procedures provide test data applicable only to the specific conditions of test. The observed value of shrinkage is influenced by so many factors that fundamental shrinkage studies (as were creep studies) are extremely complex. The influence of any one factor on the magnitude of the shrinkage movements is often obscured by the counterbalancing effect of some other variable.

The factors influencing drying shrinkage of concrete can be conveniently divided into two main categories:

(a) **Intrinsic Factors** - these relate to the internal state of the material

(b) **Extrinsic Factors** - factors such as external environment and member size which influence the internal state of the material

(c) **Secondary Effects**

(a) **Intrinsic Factors**

**Cement Type**

The influence of cement type upon shrinkage is not clearly understood. There are however a number of observations which illustrate the likely variability in shrinkage due to cement differences.

One useful observation is that the shrinkage versus drying period curve is of common form, irrespective of cement composition and fineness and is illustrated in figure 4.1.21 for OPC's produced in the UK.

The magnitude of shrinkage however, was found by one researcher to vary between cements. With SO$_3$ contents selected for minimum shrinkage the coefficient of variation was 13% and with other SO$_3$ contents the coefficient of variation was larger. A part of the variability noted by the researcher may have been due
to the difference in fineness between cements tested as other researchers suggest that an increase in fineness from 300 to 500 m²/kg may increase shrinkage by 15%. The influence of SO₃ content upon shrinkage is illustrated in Figure 4.1.22.

It is concluded that for the present purpose of shrinkage prediction cement variations will lead to a coefficient of variation of 10% in the magnitude of shrinkage.

Structure of Cement Paste

The paste content is an important factor influencing shrinkage. The water content of concrete affects shrinkage in so far as it reduces the volume of restraining aggregate and shrinkage increases with increasing water/cement ratio.

Proportion and Stiffness of Aggregate

Concrete normally contains between 55% and 80% by volume of aggregate and if the aggregate shrinks less than the paste then the aggregate restrains shrinkage. The elastic properties of the aggregate determine the degree of restraint offered. Even within the range of ordinary aggregates there is considerable variation in shrinkage. (See Figure 4.1.23)

The size and grading of aggregate themselves do not influence the magnitude of shrinkage. However, a larger aggregate permits the use of a leaner mix and means that the aggregate content can rise and hence results in a lower shrinkage. Figure 4.1.23.
Shrinkage of concrete of fixed mix proportions but made with different aggregates, and stored in air at 70°F and a relative humidity of 50 per cent.

Time reckoned since end of wet curing at the age of 28 days.

**Figure 4.1.23**

**Figure 4.1.24** (NEVILLE 12.43)*

**Figure 4.1.25** (PARROT 12.34)*

Shrinkage ratio \( \frac{S}{Sp} = \frac{\text{Shrinkage of Concrete}}{\text{Shrinkage of Neat Cement Paste}} \)

\( E_p \) & \( E_a \) are the Elastic Modulus of cement paste and aggregate respectively.

\( S_a \) = Shrinkage of aggregate.

In general, the natural aggregates used for concrete making do not shrink significantly except that in South Africa it has been found that concretes made with rocks derived from the Karroo System (Figure 4.1.26) showed shrinkage movements seven times larger than those which occur in similar concretes containing normal aggregates. The aggregates involved are shale and sandstone-like materials and...
frequently appear to be hard and sound to the degree that their use could not be condemned on appearance alone. A valuable indication of whether a particular aggregate is satisfactory or not may be provided by examination of structures which are known to have been built using that aggregate. Where there is a record of successful use over a period of at least five years, they could be used in similar structures under similar conditions of exposure, although wherever possible the aggregate should be tested as specified in SABS 1083 (12.51).

In addition dolerites are available over a large portion of the area of occurrence of shrinkage aggregates and crushed fresh dolerite is a suitable aggregate for concrete.

(b) Extrinsic Factors

Curing

Water curing increases the volume of cement gel formed and can lead to increased shrinkage of cement and concrete. However, this effect can be reversed with prolonged curing. Unfortunately as can be seen from Figure 4.1.27, these effects are also influenced by the W/C ratio of the cement paste.
The effect of curing time upon the shrinkage of concrete is illustrated in Figure 4.1.28. Shrinkage is not greatly influenced by curing times of less than a month but prolonged curing appears to reduce shrinkage without any sign of the initial increase that might be expected from the cement paste results in Figure 4.1.27.

Size and Shape of Concrete Member

Large concrete members dry more slowly than small ones and as a result for moderate drying periods shrinkage goes down with increasing member size. It is also generally accepted that ultimate shrinkage (at $t = \infty$) decreases with member size. (See Figure 4.1.29).

It is interesting to note the length of time necessary for the concrete to attain ultimate shrinkage.

Relative Humidity

Relative humidity has a major influence on the magnitude of concrete shrinkage, the shrinkage decreasing with increasing relative humidity. The influence of relative humidity upon shrinkage is illustrated in Figure 4.1.30.

(c) Secondary Effects

Admixtures

The use of certain admixtures, notably calcium chloride, and to a lesser extent lignosulphates (particularly with calcium chloride or triethanolamine additions) markedly increases shrinkage.
4.1.6 Prediction of Strains

(i) Definitions

(a) Instantaneous Strain $\varepsilon_i$

In most instances the "instantaneous strain $\varepsilon_i$" maybe assumed to be age independent and constant since in many cases of engineering practice concrete is subjected to sustained loads at an age of at least 28 days. Subsequent change in modulus of elasticity may, as an approximation, be neglected.

(NB, The author's UCT tests gave $E$ values at (i) 14 days of 29.2 GPa and (ii) 28 days of 31.7 GPa and in fact the variation between individual tests was more than the difference between the 14 and 28 days of $E$).

(b) Creep Strain

Subdivision of "creep strain" into "delayed elastic strain" and "plastic flow strain":

It is accepted that the definition "creep" is not exact since it not only consists of permanent strains but also reversible strains, see Figure 4.1.31. If the semi permanent load is removed from a test specimen then there is delayed recovery.

\[ \varepsilon_k = \text{creep strain} \]
\[ \varepsilon_v = \text{delayed elastic strain (reversible creep)} \]
\[ \varepsilon_f = \text{plastic flow strain (irreversible creep)} \]

The delayed elastic strain can only be observed when the concrete is unloaded and about half of $\varepsilon_v$ occurs within a few weeks.
The equation of creep strain is
\[ \varepsilon_k = \varepsilon_v + \varepsilon_f \]
Creep strain = delayed elastic strain + plastic flow strain

The above subdivision is important should the creep inducing load be significantly reduced at some later stage.

Subdivision of "plastic flow strain" into "instant plastic flow strain" and "remaining plastic flow strain"

An appreciable portion of the plastic flow strain occurs within a day of applying the permanent load. This instant plastic flow is especially pronounced in the case of concrete that is loaded at an early age.

The equation of creep strain becomes
\[ \varepsilon_k = \varepsilon_v + \varepsilon_a + \varepsilon_f \]
Creep strain = delayed elastic strain + instant plastic flow strain + remainder plastic flow strain

Subdivision of "remaining plastic flow strain" into "basic creep" and "drying creep"

Basic creep is the creep strain that occurs when the exchange of moisture with the surrounding atmosphere is completely prevented, while drying creep is the creep strain that occurs as a result of the drying out process.

\[ \varepsilon_f = \varepsilon_{f,g} + \varepsilon_{f, tr} \]
Remainder of plastic flow = basic creep strain + drying creep strain

Basic creep is thought to be mainly dependent of the properties of the cement paste and the moisture content of the concrete. Drying creep is thought to be similar to drying shrinkage and therefore is mainly affected by loss of moisture and size of members.
(c) Evaluation of Creep Strain Components

Delayed Elastic Strain $\varepsilon_y$

A large number of tests were evaluated to obtain the average graph of development of $\varepsilon_y$ with time (Figure 4.1.32). It was found that the values for concrete as commonly used in practice only differed marginally from the mean values even when loaded at different ages and stored at different relative humidities.

![Graph of delayed elastic strain](Image)

**FIGURE 4.1.32** *(RÜSCH 12.2)*

Development of delayed elastic strain

Instant Plastic flow strain $\varepsilon_a$

The effect of early loading on $\varepsilon_a$ is shown by the following test results:

<table>
<thead>
<tr>
<th>Age at Loading</th>
<th>1 Day $\varepsilon_a \times 10^6$</th>
<th>90 Day $\varepsilon_a \times 10^6$</th>
<th>Diff.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Instant plastic flow strain $\varepsilon_a \times 10^6$</td>
<td>2.15</td>
<td>0.25</td>
<td>1.90</td>
</tr>
<tr>
<td>Remainder of plastic strain in 3 years $\varepsilon_f \times 10^6$</td>
<td>5.75</td>
<td>4.60</td>
<td>1.15</td>
</tr>
<tr>
<td>Total plastic flow strain after 3 years $\varepsilon_f \times 10^6$</td>
<td>7.90</td>
<td>4.85</td>
<td>3.05</td>
</tr>
</tbody>
</table>

(RÜSCH 12.1)*

Remainder of plastic flow strain $\varepsilon_f$

No reliable formulas have yet been developed to allow the subdivision of the "remainder of plastic flow strain" $\varepsilon_f$ into "basic strain" $\varepsilon_{f,g}$ and drying strain $\varepsilon_{f,tr}$. However, Figure 4.1.33 shows the relative magnitude of the two components for a particular investigation where $\varepsilon_{f,g}$ and $\varepsilon_{f,tr}$ were measured.
The above creep strain curves relate to 70 x 70 x 280 mm specimens and a concrete strength of 20 MPa. The specimens were loaded at 7 days. The $\epsilon_v$ values were not measured and had to be estimated. The $\Delta \epsilon_a$ had to be introduced to allow for the effect of the difference in the curing of the specimens up to the time of loading at 7 days.

(d) **Shrinkage Strain**

As discussed in section 4.1.5, what is loosely termed shrinkage strain is in fact mostly "drying shrinkage" and is closely related to the "drying creep".

(ii) **Summarising**

A diagramatic representation of concrete strain components as well as factors influencing the strain are shown on Figure 4.1.34.

**Diagramatic representation of strain components and influencing factors**

The magnitude of the various components of concrete strain under load for a particular test series are shown in Figure 4.1.35.
Strain \( \varepsilon \) = Elastic Strain \( \varepsilon_e \) + Creep \( \varepsilon_a \) + Shrinkage \( \varepsilon_s \)

- Elastic Strain \( \varepsilon_e \) is reversible at mainly reversible after unloading.
- Creep \( \varepsilon_a \) is mainly irreversible.
- Shrinkage \( \varepsilon_s \) is partially reversible if moisture content changes.

**FIG 4.1.34**

(Rüsch 12.2)*

**DIAG. REPRESENTATION OF CONCRETE STRAIN COMPONENTS**

**FIG 4.1.35**

(Rüsch 12.2)*
4.2 CODES OF PRACTICE

4.2.1 Code Requirements

Any code of practice which seeks to give guidance to designers regarding the magnitude of creep and shrinkage must take into account as many as possible of the factors affecting creep and shrinkage eg humidity, curing and storing conditions, temperature, type of aggregate, mix proportions, water/cement ratio, size of member, and for creep, age at loading.

4.2.2 Evolution of Codes

Many of the above factors were allowed for in the design equations contained in the German Standard DIN 4227 in 1950 and which were improved on by Dr F. Leonhardt in his book "Prestressed Concrete Design and Construction" published in 1955. Later similar design equations were contained in the CEB-FIP Recommendation (12.44)* published in 1970. As a result of the experience gained with the use of the CEB-FIP 1970 recommendations, revised design equations have now been proposed in the CEB-FIP Recommendations published in 1978 (12.45)* which have yet to prove themselves.

In Britain the British Standard CP 115 (12.52)* was published in 1959 but did not take many of the factors referred to in 4.2.1 above into account, thus in hindsight making it unsuitable for use under other conditions than those prevailing in Britain, (where it generally gave satisfactory results), since such factors as humidity were not allowed for. This Code was superceeded in 1972 by British Standard CP 110 "Code of Design for the Structural use of Concrete" and in the case of bridges by BS 5400 (12.46)* in 1978 and both these codes still retain the simple approach of CP 115, but they do draw the designers attention to the fact that the values given are general and reference should be made to specialist literature i.e. the CEB-FIP Recommendations : 1978(12.44)* design equations.

American Codes have not been referred to as by tradition South Africa has followed British Practice, which now, because of the European Common Market and Metrication developments, is tending to follow the Continental Approach.
4.2.3 British Standard – CP 115 (12.52)*

The relevant parts of CP 115 (12.52)* are given in Appendix 11.7.

In this Code, for creep, firstly there is differentiated between pretensioning and post-tensioning:

(i) In the case of pre-tensioning, no limits for age at tensioning are given and the creep factor is given as $4.8 \times 10^{-6}$ per N/mm$^2$ for a concrete strength at transfer of 40 N/mm$^2$. For lower values of concrete strength at transfer the creep is increased in proportion.

(ii) For post-tensioning between 2 and 3 weeks after concreting, the creep factor is given as $36 \times 10^{-6}$ per N/mm$^2$ for concrete strength at transfer of 40 N/mm$^2$. For lower values of concrete strength at transfer creep is increased in proportion.

No allowance is made for humidity and specimen size at all. For shrinkage a value of $300 \times 10^{-6}$ is given for pre-tensioning and a value of $200 \times 10^{-6}$ for post-tensioning at 2 to 3 weeks. For post-tensioning at an earlier age an intermediate value is recommended. No allowance is made for humidity, specimen size and concrete strength.

4.2.4 CEB-FIP: 1970 (12.44)*

The relevant parts of the CEB-FIP: 1970 Recommendations are given in Appendix 11.8.

In this Code, the creep coefficients are expressed in terms of the product of five coefficients taking into account age at loading, mix proportions, specimens size and relative humidity and durations of loading. A strict application of the law of superposition was suggested to evaluate creep under variable stress. The shrinkage coefficient is also expressed in terms of five coefficients taking into account mix...
proportions, specimen size, relative humidity, percentage of steel and development of shrinkage with time.

After this Code had been used for some time a number of shortcomings became apparent.

(i) It does not show clearly that parts of the creep strain are reversible

(ii) The estimation of creep coefficients from the product of five independent variables is unsatisfactory since experimental data show some interdependence between these variables, i.e. The specimen size influences both the magnitude of creep as well as the development of creep with time and is different for different values of concrete age at the time of load application.

(iii) An estimate of stress relaxation and creep under variable loads using the CEB-FIP : 1970 method is cumbersome and sometimes resulted in doubtful answers.

4.2.5 CEB-FIP : 1978 (12.45)*

The relevant parts of the CEB-FIP : 1978 Recommendations are given in Appendix 11.9.

The creep coefficient is given as the sum of three terms which represent:

(a) The reversible part of the deformation which is developed during the first few days after the load has been applied

(b) The recoverable part of the delayed deformation (delayed elasticity) assumed to be independent of aging in its development

(c) The irreversible delayed deformation (flow) which is very much affected by the age at which the loading commences.

The shrinkage coefficient is given as the product of three coefficients which are dependent on ambient humidity, specimen size, mix design and ambient temperature.
As with any new approach, there are those specialists who advocate its use, in this case Rüsch (12.10)* while on the other hand there are those who oppose it, in this case Bazant (12.10 a & c) who uses optimization techniques to show that in contrast to the CEB-FIP: 1970 creep function, the form of the CEB-FIP: 1978 creep function cannot reasonably approximate experimental creep curves over the full range of time intervals of interest while Rüsch counters this in (12.10(b)) by pointing out that for practical engineering conditions the CEB-FIP: 1978 creep function does in fact approximate the test data.

4.2.6 BS 5400: 1978 (12.46)*

The relevant parts of BS 5400: 1978 are given in Appendix 11.9.

This Code is a great improvement on CP 115. It retains for the general cases the simple approach of CP 115, which in the case of shrinkage (but not creep) has been improved by giving factors for two different relative humidities. However, no allowance is made for member size.

At the end of the shrinkage and creep section of the Code, it is stated that "in applying the recommendations of the Code, which are necessarily general, reference should be made to specialist literature for more detailed information on the factors affecting shrinkage and creep". This information is given in Appendix C "Shrinkage and Creep" of the Code and is similar to the information given in CEB-FIP: 1970 recommendations.

4.2.7 SABS 0100: PART 1: 1980 (12.68)*

The clauses of this Code referring to elastic, creep and shrinkage strain are the same as the "general" clauses of BS 5400: 1978 except that in the case of shrinkage strain, allowance has been made for different relative humidities prevailing in South Africa i.e.

<table>
<thead>
<tr>
<th>System</th>
<th>Shrinkage per Unit Length</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rel. H 80%</td>
</tr>
<tr>
<td>Coastal Town</td>
<td></td>
</tr>
<tr>
<td>Most Inland Areas</td>
<td></td>
</tr>
<tr>
<td>Low Rel. H i.e. Windhoek &amp; Upington</td>
<td></td>
</tr>
<tr>
<td>Post Tensioning transfer at 7 to 14 days after concreting</td>
<td>$140 \times 10^{-6}$</td>
</tr>
</tbody>
</table>
4.3 TEMPERATURE MOVEMENTS AND STRAINS

4.3.1 General
Depending on the measuring devices used, the strains measured on actual structures can be significantly affected by thermal stresses which change continually. In contrast, these effects can be completely eliminated in the laboratory by keeping the test specimens at a constant temperature for the duration of the testing program.

4.3.2 Temperature effects
Temperature movement in bridges consists of axial movement along the neutral axis when the temperature is uniform and constant throughout the deck and axial and rotational movement due to thermal bending, when there is a vertical temperature gradient through the deck.

The thermal response of bridge decks is a complex transient phenomenon influenced by many factors, as indicated for the typical box section shown below.

FIG. 4.3.1—FACTORS AFFECTING THERMAL RESPONSE

(PRIESTLEY 12.30)*
4.3.3 Vertical Temperature Gradient

In simply supported members internal strains are set up due to the vertical temperature gradients that are in equilibrium with each other. In continuous structures additional moments and strains result from the fact that the deck either wants to hog upwards (or deflect downwards) between the abutments and is prevented from doing so by the internal supports.

Elimination of Temperature Gradient Effects

If the strain measurements on an actual structure can be made at a time of the day when the temperature is approximately uniform throughout its depth, then only axial strains have to be corrected for, thereby greatly simplifying the correction required.

Research has been initiated in many countries into the temperature effects on structures, and the findings of the following research indicate at which time of the day the temperature is approximately uniform throughout a superstructure:
(i) GREAT BRITAIN

In Great Britain long term measurements have been made (Emerson - 12.29)* in order to correlate the range of bridge temperature with range of shade temperature in the same area and it was found for concrete bridge that:

(a) over a 24 hour period, the mean temperature of the superstructure is lowest at 08.00 ± 1 hour (GMT) and the overall length is a minimum.

(b) the mean superstructure temperature at 08.00 ± 1 hour (GMT) for each day is usually within ± 3°C of the air temperature in the shade at the same time.

(c) also at this time, the temperature is approximately uniform throughout the superstructure (Tyler - 12.13).*

(d) owing to the thermal mass of a structure there is, over a 24 hour period, no satisfactory correlation between the instantaneous shade temperature and the instantaneous mean bridge temperature, except for the approximation at 08.00 ± 1 hour (GMT)

(e) if the structure is a box construction, the instantaneous air temperature measured inside the box does not represent the true instantaneous mean bridge temperature; during the day it is usually both below, and out of phase with the mean bridge temperature.
(ii) NEW ZEALAND

Over the past few years research has been conducted into the structural behaviour of concrete bridges.

The following temperatures were recorded over a two day summer period at mid span (Section A-A) of a two span continuous box bridge deck (Priestley - 12.5)*

At 08.00 hour the recorded temperatures are all within 18 + 2°C which indicate that the temperature is essentially uniform and constant throughout the depth of the cross-section.
(iii) SOUTH WEST AFRICA

In South West Africa, research projects relating to the effects of climate on the design and performance of road pavements included the instrumentation of experimental road pavements, automatic collection and computer analysis of resulting temperature data. For a experimental site in Ovambo, the following temperatures were recorded (Williamson-12.28):

![Graph showing temperature profiles at different depths](image)

**Fig 6:** Data from an experimental site in Ovambo (SWAL)

**FIG. 4.3.4** (WILLIAMSON 12.28)
From the recorded temperature it can be seen that at 09.00 hour (S.A. time) the temperature was practically uniform throughout the 300 mm depth of pavement investigated and the surface temperature approximated most closely with the pavement temperature.

(iv) **ON SITE INVESTIGATIONS**

In order to ascertain whether the findings of the various researchers applied to the Brackwater South Bridge it was decided to read all the gauge points at section 0.75, at one hourly intervals for a day.

The results of the readings on a day in January are recorded in Appendix 11.5 and all the gauge points show that up to 08.00 hours (S.A. time) the bridge was shortening and after 10.00 hours (S.A. Time) the bridge was expanding. At this time both gauge points at underside of top slab and topside of bottom slab changed from -ve strains to +ve strains confirming that at this stage the temperature is approximately uniform and constant through the deck.

(v) **CONCLUSION**

Based on the finding of researchers, (which were later confirmed by our own measurements) it was decided that the readings should be taken early in the mornings after sunrise, but before the sun had warmed up the bridge to ensure that only axial temperature strains have to be corrected for.
The coefficient of thermal expansion of concrete depends on the composition of the mix and on its state of hydration at the time of the temperature change.

(i) **Mix Proportions**

The influence of the mix proportions arises from the fact that the three main constituents of concrete, the cement paste, the fine and coarse aggregate, (as they may be different aggregates), all have dissimilar thermal coefficients and the coefficient of the concrete is a resultant of the three values.

<table>
<thead>
<tr>
<th>Type of rock</th>
<th>Average coefficient of thermal expansion $\times 10^{-6}^\circ$C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartzite, silica shale, cherts</td>
<td>11.0-12.5</td>
</tr>
<tr>
<td>Sandstones</td>
<td>10.5-12.0</td>
</tr>
<tr>
<td>Quartz sands and pebbles</td>
<td>10.0-12.5</td>
</tr>
<tr>
<td>Clays and mica shales</td>
<td>9.5-11.0</td>
</tr>
<tr>
<td>Granites and gneisses</td>
<td>6.5-8.5</td>
</tr>
<tr>
<td>Syenites, feldspathic porphyry,</td>
<td></td>
</tr>
<tr>
<td>diorites, andesite, phonolite,</td>
<td></td>
</tr>
<tr>
<td>gabbros, diabase, basalt</td>
<td>5.5-8.0</td>
</tr>
<tr>
<td>Limestones</td>
<td>3.5-6.0</td>
</tr>
<tr>
<td>Pure calcite</td>
<td>4.0-6.5</td>
</tr>
<tr>
<td>Marbles</td>
<td>4.0-7.0</td>
</tr>
<tr>
<td>Dolomites, magnesites</td>
<td>7.0-10.0</td>
</tr>
</tbody>
</table>

**FIG. 4·3·5**

(FULTON 12·57)
(ii) **Moisture Condition**

The influence of the moisture condition applies to the paste component and is due to the fact that the thermal coefficient is made up of two movements; the true kinetic thermal coefficient and swelling pressure. Swelling pressure arises from a decrease in the capillary tension of water held by the paste with an increase in temperature, and the coefficient itself decreases with age due to a reduction in the potential swelling pressure owing to an increase in the amount of crystalline material in the hardened cement paste.

![Coefficient of Expansion vs Relative Humidity](image)

**THE LINEAR COEFFICIENT OF THERMAL EXPANSION OF NEAT CEMENT PASTE AT DIFFERENT AGES**

Figure 4.3.6 refers to neat cement and although the effects are also apparent in concrete, the variation in the coefficient is much smaller since it is only the paste component that is affected by the relative humidity and ageing.

(iii) **Research Data**

Overseas research data is shown in Table 4.3.7 which gives values of the thermal coefficient of concrete made with different types of aggregates and cured under various conditions.
COEFFICIENT OF THERMAL EXPANSION OF 1:6 CONCRETES MADE WITH DIFFERENT AGGREGATES

<table>
<thead>
<tr>
<th>Type of aggregate</th>
<th>Linear coefficient of thermal expansion $10^{-6}$ per °F</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Air-cured concrete at 64% Rel. Hum.</td>
</tr>
<tr>
<td>Gravel</td>
<td>7.3</td>
</tr>
<tr>
<td>Granite</td>
<td>5.3</td>
</tr>
<tr>
<td>Quartzite</td>
<td>7.1</td>
</tr>
<tr>
<td>Dolerite</td>
<td>5.3</td>
</tr>
<tr>
<td>Sandstone</td>
<td>6.5</td>
</tr>
<tr>
<td>Limestone</td>
<td>4.1</td>
</tr>
<tr>
<td>Portland stone</td>
<td>4.1</td>
</tr>
<tr>
<td>Blast-furnace slag</td>
<td>5.9</td>
</tr>
<tr>
<td>Foamed slag</td>
<td>6.7</td>
</tr>
</tbody>
</table>

**TABLE 4.3.7**

The results of local research is shown in Table 4.3.8 and although the condition of curing is not specified this is not important as can be seen from Table 4.3.7 above.

**EFFECT OF AGGREGATE TYPE ON COEFFICIENT OF THERMAL EXPANSION OF CONCRETE**

<table>
<thead>
<tr>
<th>Aggregate type</th>
<th>Coefficient of thermal expansion of concrete $\times 10^{-6}°C$</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Granites and rhyolites</td>
<td>6.8-9.5</td>
<td>7, 10</td>
</tr>
<tr>
<td>Sandstones</td>
<td>11.7</td>
<td>7</td>
</tr>
<tr>
<td>Quartzites</td>
<td>12.8</td>
<td>7</td>
</tr>
<tr>
<td>Limestones</td>
<td>6.1-9.9</td>
<td>7, 9, 10</td>
</tr>
<tr>
<td>Marble</td>
<td>4.1</td>
<td>9</td>
</tr>
<tr>
<td>Dolerite</td>
<td>9.5</td>
<td>7</td>
</tr>
<tr>
<td>Quartz</td>
<td>10.4</td>
<td>9</td>
</tr>
<tr>
<td>Blastfurnace slag</td>
<td>10.6</td>
<td>7</td>
</tr>
<tr>
<td>Witwatersrand quartzite</td>
<td>12.2</td>
<td>17</td>
</tr>
<tr>
<td>Granite from North of Johannesburg (Jukkasjärvi)</td>
<td>9.4</td>
<td>17</td>
</tr>
<tr>
<td>Dolomite from Olifantsfontein</td>
<td>8.6</td>
<td>17</td>
</tr>
<tr>
<td>Malmsbury hornstone from Cape Peninsula</td>
<td>10.9</td>
<td>17</td>
</tr>
<tr>
<td>Limestone (50/50 by mass Lichtenburg/ Ubot)</td>
<td>9.7</td>
<td>17</td>
</tr>
<tr>
<td>Namaqualand onyx</td>
<td>10.3</td>
<td>17</td>
</tr>
<tr>
<td>Dolerite from the Orange Fish Tunnel</td>
<td>7.5</td>
<td>17</td>
</tr>
<tr>
<td>Felsite from Witbank</td>
<td>9.2</td>
<td>17</td>
</tr>
</tbody>
</table>

**TABLE 4.3.8**

For concrete made with Quartzite aggregate, the coefficient of Thermal Expansion, based on research data is therefore of the order of $12 \times 10^{-6}$ per °C.
(iv) Actual Experimental Investigations

In order to confirm that the coefficient of thermal expansion as determined from a literature survey was applicable to the concrete of the Brackwater South Bridge it was decided to confirm the coefficient experimentally.

Two of the four shrinkage control cylinders were removed from the constant temperature (24°C) and constant humidity (50% Rel Hum) and placed in an oven and heated to 34°C and the change in length determined. For practical reasons it was not possible to raise the temperature in the constant humidity room and an oven had to be used instead, but the effect of drying out of the concrete should be small in view of the fact that concrete cylinders are already 2½ years old. The results of the experiment are recorded in Appendix 11.6 and show that experimental coefficient of thermal expansion of the concrete is $12 \times 10^{-6} \, ^\circ C$.

Conclusion
Based on the finding of other researchers and our own measurements it was decided to use a coefficient of thermal expansion of $12 \times 10^{-6}$ per °C

In order to eliminate temperature and also seasonal moisture swings as far as possible readings are required for at least 12 month intervals and in fact were carried on for 3 years.
4.4 MEASUREMENT OF STRAIN AND STRESS IN CONCRETE STRUCTURES

4.4.1 General

In order to investigate the magnitude of elastic creep and shrinkage deformations of actual concrete bridges as well as laboratory specimens it is necessary either to measure stresses or strains. While stress gauges do exist, they are usually expensive, unreliable or inconvenient to use while strain gauges are more practical and the "method of identical strains" has been used to estimate stresses from measured strains (Tyler 12.17)*.

4.4.2 Strain Gauges

In the research carried out, two types of strain gauges were generally used i.e. the acoustic strain gauge and the demec demountable gauge.

(i) Acoustic Strain Gauges

Principle of operation.

Essentially the gauge consists of a wire stretched between two anchorage points which are fixed to the member under investigation. A change in the distances between the anchorages causes a corresponding strain change in the wire and a change in the frequency of vibration. An electromagnet at the centre both plucks the wire and transmits the frequency to a recording device, which either indicates the frequency of vibration, or the time required to complete a given number of cycles.

The Vibrating Wire

A silver plated piano wire 0.0254 mm in diameter or similar is used in acoustic gauges. The wire is heat-treated by the gauge manufacturer to obviate creep at elevated temperature, in case the
gauges are used in nuclear pressure vessels or similar.

The value of the coeff of expansion of the wire may be taken as 12,0 (± 0,5) x 10⁻⁶ per °C. This is an important characteristic of the gauge, since it roughly corresponds with the coeff of expansion of concrete and steel used in bridge construction. Thus temperature changes cause the wire to expand with the structure for buried gauges or surface gauges protected by covers, and in general, only elastic strain, creep and shrinkage are recorded by the gauge.

Accuracy of Readings

The accuracy of the strain readings depends mostly on the resolving power of the recording equipment and is of the order of ± 1,5 to ± 3 x 10⁻⁶.

Recording Equipment

Generally consists of battery operated portable equipment in which the vibration from the gauges is compared electronically with that of a standard source (a quartz crystal) and a digital display indicates the time for the wire to perform a fixed number of cycles. The strain in the wire is then obtained by referencing to tables incorporating the appropriate gauge factor.

RRL. Burried Strain Gauges for Concrete

The gauge wires are enclosed in a stainless steel barrel to prevent damage during concreting (See Figure 4.4.1). The gauges are installed in a structure before concrete is placed by tying them to the nearest reinforcing bar.

RRL Surface Strain

The surface acoustic strain gauge as shown in Figure 4.4.2 has been developed to reliably measure long-term changes in strain at the surface of bridges.
Steel-barrelled R.R.L. acoustic gauge

**FIGURE 4.4.1**  
(TYLER 12.17)*

![Diagram of Steel-barrelled R.R.L. acoustic gauge]

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**SURFACE ACOUSTIC GAUGE**

**FIGURE 4.4.2**  
(TYLER 12.17)*

(ii) **Demec Demountable Strain Gauge**

**Principle of Operation**

In the demec demountable strain gauge (see Figure 4.4.3) two virtual point connections are used between the instrument and the specimens and translational freedom between the connections is provided by a moving...
arm pivoted about a knife edge and seating. The practical form of connection used is that of a cone locating in a cylindrical hole.

The Reference Discs

It has been found that simple mild steel locating discs give as good results as elaborately treated ones.

Care is required in fixing the discs to the concrete with a quick setting epoxy, the exact spacing of the two discs being achieved by a setting-out gauge bar.

Holding the Gauge

It is important that the pressure applied is just sufficient to provide good contact, any increase causes wear of the discs and of the instrument.

The instrument should always be used the same way round on a particular gauge length, as on the Invar dummy bar.
The Invar Dummy Bar

Although it may be argued that, in conditions where the concrete is going to change in temperature during testing, a concrete dummy bar, which will behave in a similar manner to the structure is required, the practical difficulties are considerable. It has been found that it is better to eliminate the temperature effect on the instrument by checking against an Invar bar and to allow separately for known temperature changes in the structure.

Accuracy of Readings

With due care, the gauge is accurate to ± 1 division on the dial. The sensitivity of the instrument increases with the gauge length employed, and for the standard 200 mm gauge length the strains can be read to an accuracy of ± 3 x 10⁻⁶.

(iii) Comparison of the Usage of Demountable vs Acoustic Gauges

The acoustic gauge has been extensively used because:

(a) It can be used remotely and leads can be taken to a central place for readings

(b) A large number of gauges can be read quickly with portable equipment

(c) Since the coefficient of expansion of concrete is similar to that of the wire of the gauge, readings of creep and shrinkage from buried gauges may be taken throughout the year without temperature corrections.

The Demec gauge is most useful for short term tests such as proof testing of a bridge structure using a static vehicle load, when usually no temperature correction is required as a new zero is used for each test.

The Demec gauge is also useful for tests in the laboratory where a large number of readings are required and also for the calibration of new types of gauges. In fact the reliability of the acoustic
strain gauges as a means of measuring long-term strains was established by comparing results from Demec gauges used at the surface of the concrete for the Medway Bridge (Tyler 12.12)* with those from buried acoustic strain gauges. (The Demec readings were temperature corrected).

One important consideration when choosing a gauge is cost. The capital outlay for a Demec gauge is less than one tenth of that for a portable acoustic gauge recording equipment. In addition, Demec gauge points cost very little in comparison with acoustic gauges even when recoverable surface gauges are employed.

4.4.3 Stress Gauges

Stresses may either be determined directly in the field by means of a stress gauge or alternatively may be computed from measured strains.

(i) Direct Stress Measurement

The devices which do exist for the direct measurement of stress in concrete structures are either expensive or have disadvantages when used in structure in the field. Two types that have been used in research on bridges are:

(a) Glotzl Pressure Cell

Principle of Operation

The cell consists of a flat steel envelope connected by a tube (See Figure 4.4.4) to a valve chamber which is initially filled with a fixed quantity of oil which is sealed off in the chamber by a membrane which operates a valve in the external pumping circuit.
When pumping is commenced the pressure in the valve chamber and envelope is raised. When the pressure in the envelope is equal to the applied external stress, a small volume change in the envelope opens the valve and allows oil to flow freely round the external circuit and there is no further pressure rise. The maximum recorded stress for the pumping speed specified by the manufacturer is thus the stress in the concrete.

Record of Performance

Glotzl cells have been used in the box girders of three major structures (Tyler 12.14)* and gave satisfactory stress readings. The cost of the cell is about five times as much as the cost of an acoustic strain gauge.

(b) Photoelastic Stress Plugs

Photoelastic stress plugs of the glass type have been used on a limited scale in research (Tyler 12.14)* and they have to be viewed with a light source in order to take readings and one disadvantage of the plug is that some experience is required in interpolating readings in a biaxial stress field. An advantage of it is that the plug will indicate the directions of principal stresses.
(ii) Indirect Method of Estimating Stress using the Method of Identical Strains in Control Specimens

Basic of Method

Essentially this method consists of loading control specimens placed in creep rigs in exposure conditions similar to those existing at the site to give the same strain as measured in the full-sized structure. The stresses on the specimen, which can be calculated from the load applied to the creep rig are then the same as in the structure.

Creep Rig

A portable creep rig as shown in Figure 4.4.5 has been developed in order to load small specimens under the same conditions of exposure as the bridge structure.

The rig basically consists of two upper and a lower baseplate joined by three high tensile steel rods. Load is applied by means of an hydraulic jack inserted between the upper two baseplates. The nuts above the intermediate plate are then tightened and the jack load released.

The strains in the concrete specimen held between the intermediate plate and the lower baseplate is measured by a single acoustic gauge located at the centre of the specimen.

![Creep Rig Diagram](image-url)
Site Conditions

A bridge is subjected to temperature and humidity conditions which vary from day to day and season to season, and as both temperature and humidity affect the magnitude of inelastic movements the specimens must be placed in an outside exposure hut to give roughly similar conditions to the specimens as those prevailing at the bridge site, i.e. protection from rain as provided by the bridge waterproofing, together with a free circulation of air.

Site Effect

The size and shape of a concrete member influences creep and shrinkage behaviour and the specimens must be waterproofed in such a way as to eliminate the size effect. It has been found experimentally (Tyler 12.16)* that this is achieved if specimens are waterproofed to give the same volume surface area ratio as the bridge by leaving two untreated bands near each end of the specimen (See Figure 4.4.6). in such a way that the length of the treated portion at the centre of the specimen containing the acoustic gauge is equal in thickness to that of the structural element under test. In this way the critical length of the moisture path from the centre of the specimen to the free surface of the concrete is roughly the same as for the structure.

![Simulation of slab of depth 4d in small specimen for loading in rig](image)

**FIGURE 4.4.6 (TYLER 12.16)***
Strain Measurement

The acoustic strain gauge has been found to be the most convenient gauge to measure the strains in both structure and the specimens. However, a much cheaper, although less convenient method is to measure the strains with a Demec gauge.

4.4.4 CONCLUSION

From the foregoing it is clear that:

(i) If strains have to be measured in a structure, then buried acoustic strain gauges are the most convenient to use, but the equipment is expensive.

(ii) Alternatively Demec gauges can be used to measure strains, they are as accurate, but much cheaper although inconvenient to use.

(iii) If stresses in the structure are required then the method of "identical strains" as described give the best results.

For the site strain measurements carried out as part of this investigation the Demec gauge had to be used as it was not possible to obtain the use of acoustic gauge recording equipment. Unfortunately the determination of actual stresses in the structure was beyond the scope of this investigation.
4.5 OBSERVED STRAINS IN BRIDGES

4.5.1 General

Many characteristics of concrete bridge behaviour cannot adequately be measured under laboratory conditions because of dependence on size and ambient conditions. Creep and shrinkage, and response to ambient temperature fluctuations, fall into this category.

The theories developed to predict the different aspects of concrete bridge behaviour must therefore be evaluated and tested by comparison with observed measurements on actual structures.

4.5.2 German Investigations

(i) Object

With the introduction of the Dywidag-Prestressing System the firm of Dyckerhoff and Widman decided to observe the behaviour of prestressed concrete structures in order to check how realistic the theoretical values of creep and shrinkage movements based on DIN 4227 - "Spannbeton, Richtlinien für Bemessung und Ausführung (12.60)" were.

(ii) Structures Instrumented

Under the supervision of Dr Finsterwalder (12.7) the following bridges which were prestressed using the Dywidag Prestressing system were instrumented during the construction stage and the deformations continually recorded and analysed:

<table>
<thead>
<tr>
<th>Bridges</th>
<th>Year of Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Luitpoldbrücke Landshut</td>
<td>1950</td>
</tr>
<tr>
<td>Günstorbrücke Über die Donau, Ulm</td>
<td>1950</td>
</tr>
<tr>
<td>Lechauser Strassenbrücke Augsburg</td>
<td>1950</td>
</tr>
<tr>
<td>Werderbrücke Pforzheim</td>
<td>1950</td>
</tr>
<tr>
<td>Rheinbrücke Worms</td>
<td>1951</td>
</tr>
<tr>
<td>Neue Königsdammbrücke Berlin</td>
<td>1952</td>
</tr>
</tbody>
</table>
Bridges (Cont.)

Lombardsbrücke Hamburg 1952
Brudermühlbrücke München 1953
Eisenbahnbrücke Horrem 1953
Moselbrücke Koblenz 1953
Hochstrasse Unkelstein 1956

Buildings

Deckenbinder der Berliner Bank, Berlin 1952
Fabrikgebäude Steiff, Giengen/Benz 1953

(iii) Instrumentation

The shortening of the bridge superstructure was and is still measured by means of an unstressed steel rod in a pipe and cast into the concrete. The steel rod can slide longitudinally in the pipe and is fixed at the one end. At the free end of the steel rod a displacement of the end of the steel rod relative to the surrounding concrete occurs due to the elastic, creep and shrinkage shortening of the concrete, which is recorded by a measuring device. The location of such an unstressed steel rod in a superstructure is shown in Figure 4.5.1 and the measuring device in Figure 4.5.2.

![Diagram of bridge superstructure and measurement device](image)

FIGURE 4.5.1

LOCATION OF UNSTRESSED STEEL ROD IN A DECK

The measuring device is connected to the steel rod by means of a steel tape and records the changes in length of the superstructure at the position of the steel rod. The relative displacement of the end of the steel rod relative to the surrounding concrete at the free end...
is independent of temperature changes because the coefficient of thermal expansion of steel and concrete are approximately the same and the temperature of the steel rod is the same as that of the surrounding concrete. Thus the recorded shortening of concrete is basically only due to elastic shortening, creep and shrinkage of concrete.

(iv) Evaluation of Observed Movements

The results of the investigations are shown on diagrams 1 to 12 of (12.7b)* which are reproduced in Appendix 11.12. The top graph of each diagram shows the observed "change in length" of the steel rod in each month and the bottom graph shows the creep and shrinkage movements that have been observed to date as a % of the theoretical final value of creep and shrinkage.

The research has shown the following:

(a) The readings for all the bridges show a similar pattern and it has been found that the bridges shorten in the spring and summer and then slightly expand or remain static in the winter months. This is due to the moisture change of the concrete during the different seasons.

(b) The seasonal variation of movement does not occur in prestressed beams of the two buildings under observation as the beams are protected from the elements and the humidity remains constant.
throughout the year.

(c) For the bridges constructed in 1950 and 1951 most of the creep and shrinkage has taken place after 8 years. The total theoretical creep and shrinkage movement was based on a creep value of $k_{\infty} = 0.75 \times 2.0 = 1.5$ and a shrinkage value of $k \times e_0 = 0.75 \times 15 \times 10^{-5}$

$$= 11.25 \times 10^{-5}$$

The actual measured creep and shrinkage movement only reached 75% of this value.

(d) The development of creep depends on the time of the year when the prestressing is applied. For the "Eisenbahnbrücke Horren", which was prestressed during a wet summer, the initial creep was considerably less than for the Brudermühlbrücke München which was prestressed in a dry summer.

(v) Summary

The observations show that the Theory and Practice agree well and that the theoretical values based on DIN 4227 (12.60)* are in general on the safe side.

(vi) Addendum

In a later evaluation of Field Tests and Laboratory Results Jungwirth (12.25)* has found that the CEB-FIP:1970 Code (12.44)* recommendations for the theoretical determination of creep and shrinkage gave values which agreed within ± 15% of the values observed on the actual bridges.

4.5.3 Austrian Investigations

(i) Object

It had been observed earlier (Finsterwalder 12.7)* that the creep and shrinkage of bridges is influenced by the season they are constructed in. In order to obtain more information about the effect Aichhorn (12.18)* and (12.62)* decided to carry out further investigation.

(ii) Structures Instrumented

Three bridges (i.e. (i) Kremsdorfbrücke; (ii) Aitertalbrücke and (iii) Almbrücke) were instrumented and the superstructure shortening recorded for a number of years. The concrete used for all three bridges was similar and the bridges were located in close proximity to each other in an area with similar climatic conditions. Thus the observed values
from each bridge could be compared directly without introducing significant errors.

(iii) Instrumentation

The measuring device consisted of an unbounded 30m long steel rod embedded in the concrete of the superstructure and fixed at one end only. The displacement of the free end of the steel rod relative to the surrounding concrete was the movement of the superstructure.

(iv) Theory

The creep and shrinkage is influenced by many factors such as;
(a) composition of concrete; (b) weather especially humidity and temperature and it is therefore possible to approximately compare the observed movement of concrete bridges constructed in different seasons.

The theoretically expected total shortening in diagrams referred to as 100% was calculated assuming creep factor $\varphi = 1.625$ and a shrinkage value after the 1st prestressing (80% prestressing) of $\varepsilon = 0.6 \times 10^{-5} = 6 \times 10^{-5}$. As a comparison the development curve of creep vs time was plotted according to Leonhardt (12.60)*.

(v) Evaluation of Observed Movements

Analysis of the various graphs of Figure 4.5.3 shows that:
(a) Spring Concrete (April) (Almbrücke - Bauabschnitt 3 - Süd)

The curve of observed creep and shrinkage approximately follows the theoretical curve of creep, it increases in summer and falls off in winter.

(b) Summer Concrete (July) (Aiterstalbrücke - Nord)

As a result of the dry weather in July, the concrete is already dried out so that it only creeps slowly.

In the next month the damp weather inhibits the creep and shrinkage so that in September only a relatively small amount of creep and shrinkage have occurred. With the onset of the damp Autumn weather the concrete begins to swell as a result of the increase in capillary water, which nearly cancels the creep movements. Next spring with the warm and dry weather the creep movements increase again. As a result the loss of prestress is less than calculated.

(c) Autumn Concrete (October) (Almbrücke - Bauabschnitt 2 - Süd)

The development of movement is similar to the summer concrete.
(d) **Winter Concrete (January) (Aitertalbrücke - Süd)**

High humidity in the time interval between casting the concrete and applying the prestress prevents the concrete from drying out. The shrinkage remains small and swelling does not occur. Then the creep due to prestressing becomes more significant.

(e) **Winter Concrete (January) (Kremsdorfbrücke)**

The observed curve of creep and shrinkage is similar to that of the winter concrete of the Aitertalbrücke - Süd above.

(vi) **Summary**

All concrete shows a definite increase in the creep and shrinkage movements in the summer months and a decrease during winter months. This phenomenon can be explained that in summer due to the drying out of the structure a shortening occurs while in winter due to absorption of water a swelling of the structure occurs.

In principle the small creep and shrinkage movements of the summer concrete are due to the rapid hardening of the concrete. As a result the total actual creep and shrinkage of the summer concrete will therefore be less than the theoretical values while the actual values of the winter concrete will generally exceed the theoretical values.

The observations further showed that creep and shrinkage still increasing and with the exception of the summer concrete in fact exceeds the theoretical values.

4.5.4 **British Investigations**

(i) **Object**

The scarcity of information on the effects of creep shrinkage and temperature on the behaviour of actual bridges in Britain prompted the Road Research Laboratory to undertake measurements on certain bridges built in Britain since 1962.

(ii) **Structures Instrumented**

Under the supervision of Dr Tyler (12.1)* a representative selection of concrete bridges, mainly prestressed, were instrumented. The bridges tested are listed in Table 4.5.4(a) and the concrete mixes used in Table 4.5.4(b).
The bridges discussed in the paper.

<table>
<thead>
<tr>
<th>Structure</th>
<th>Date</th>
<th>Type of structure ((P =) precast, (I =) in situ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Medway Bridge (M2)</td>
<td>1963</td>
<td>Prestressed multiple box girder (I)</td>
</tr>
<tr>
<td>Chiswick cantilever (M4)</td>
<td>1964</td>
<td>Plain reinforced concrete (I)</td>
</tr>
<tr>
<td>Sutton Lane Bridge (M4)</td>
<td>1964</td>
<td>Prestressed multiple box girder (P and I)</td>
</tr>
<tr>
<td>River Ogmore Bridge, Bridgend, Glamorgan</td>
<td>1964-5</td>
<td>'Prelux' composite steel and concrete beam (P)</td>
</tr>
<tr>
<td>River Aire Bridge (A1M)</td>
<td>1965</td>
<td>Prestressed multiple box girder (I)*</td>
</tr>
<tr>
<td>Mancunian Way</td>
<td>1965-6</td>
<td>Prestressed segmental concrete box girder (P)</td>
</tr>
<tr>
<td>Elizabeth Bridge, Windsor</td>
<td>1966</td>
<td>Concrete beams (P) with deck slab (I)</td>
</tr>
<tr>
<td>Western Avenue (M40)</td>
<td>1968</td>
<td>Multiple segmental concrete box girder (P)</td>
</tr>
<tr>
<td>Loudwater Viaduct (M40)</td>
<td>1968</td>
<td>Steel beams composite with concrete deck (I)</td>
</tr>
<tr>
<td>London Bridge</td>
<td>1968-9</td>
<td>Multiple segmental box girder (P)*</td>
</tr>
<tr>
<td>Braidley Road Bridge, Bournemouth</td>
<td>1969</td>
<td>Prestressed multiple box girder (I)</td>
</tr>
<tr>
<td>Redesdale Bridge, Northumberland</td>
<td>1969</td>
<td>Prestressed 'Lytag' beams (P) in slab deck (I)</td>
</tr>
</tbody>
</table>

*Samples of concrete only tested by the Laboratory.

Details of concrete mixes (proportions are listed per unit quantity of cement).

<table>
<thead>
<tr>
<th>Bridge or structure</th>
<th>In situ</th>
<th>Type of cement*</th>
<th>Sand Source</th>
<th>Coarse aggregate</th>
<th>Over-all</th>
<th>Water cement</th>
<th>Typical values of modulus of elasticity at 28 days (kN mm(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(I)</td>
<td>(P)</td>
<td>Type</td>
<td>Proportion</td>
<td>Type</td>
<td>Proportion</td>
<td>ratio</td>
</tr>
<tr>
<td>Medway Bridge</td>
<td>I</td>
<td>O.P.</td>
<td>pit</td>
<td>1 00</td>
<td>flint</td>
<td>0·68</td>
<td>2·32</td>
</tr>
<tr>
<td>Chiswick cantilever</td>
<td>I</td>
<td>O.P.</td>
<td>pit</td>
<td>2 20</td>
<td>flint</td>
<td>3·37</td>
<td>5·57</td>
</tr>
<tr>
<td>Bison beams(^{111})</td>
<td>P</td>
<td>R.H.P.</td>
<td>pit</td>
<td>1 01</td>
<td>flint</td>
<td>0·53</td>
<td>1·97</td>
</tr>
<tr>
<td>Loudwater Viaduct</td>
<td>I</td>
<td>O.P.</td>
<td>pit</td>
<td>2 00</td>
<td>flint</td>
<td>3·0</td>
<td>5·00</td>
</tr>
<tr>
<td>Sutton Lane Bridge</td>
<td>P, I</td>
<td>O.P.</td>
<td>pit</td>
<td>0·75</td>
<td>flint</td>
<td>0·68</td>
<td>2·32</td>
</tr>
<tr>
<td>Preflex beams</td>
<td>P</td>
<td>R.H.P.</td>
<td>pit</td>
<td>1 00</td>
<td>flint</td>
<td>1·08</td>
<td>1·60</td>
</tr>
<tr>
<td>River Aire Bridge</td>
<td>I</td>
<td>O.P.</td>
<td>river</td>
<td>1·43</td>
<td>quartzite</td>
<td>0·91</td>
<td>1·46</td>
</tr>
<tr>
<td>Mancunian Way</td>
<td>P</td>
<td>O.P.</td>
<td>river</td>
<td>0·71</td>
<td>limestone</td>
<td>0·95</td>
<td>1·89</td>
</tr>
<tr>
<td>Elizabeth Bridge</td>
<td>P</td>
<td>R.H.P.</td>
<td>river</td>
<td>1·00</td>
<td>flint</td>
<td>1·08</td>
<td>1·60</td>
</tr>
<tr>
<td>Western Avenue</td>
<td>I</td>
<td>O.P.</td>
<td>river</td>
<td>1·32</td>
<td>flint</td>
<td>2·08</td>
<td>3·40</td>
</tr>
<tr>
<td>London Bridge</td>
<td>P</td>
<td>O.P.</td>
<td>river</td>
<td>0·93</td>
<td>flint</td>
<td>0·54</td>
<td>1·78</td>
</tr>
<tr>
<td>Braidley Road Bridge</td>
<td>I</td>
<td>O.P.</td>
<td>river</td>
<td>1·65</td>
<td>granite</td>
<td>0·86</td>
<td>1·96</td>
</tr>
<tr>
<td>Lytag beams</td>
<td>P</td>
<td>R.H.P.</td>
<td>river</td>
<td>1·05</td>
<td>limestone</td>
<td>2·45</td>
<td>3·50</td>
</tr>
<tr>
<td>Solite specimens</td>
<td>P</td>
<td>R.H.P.</td>
<td>river</td>
<td>0·57</td>
<td>Lytag</td>
<td>1·33**</td>
<td>2·90</td>
</tr>
<tr>
<td>Exposure slabs</td>
<td>I</td>
<td>O.P.</td>
<td>river</td>
<td>1·00</td>
<td>Solite</td>
<td>1·12††</td>
<td>2·12</td>
</tr>
</tbody>
</table>

(iv) Evaluation of Observed Creep Strains

(a) Observed results on full scale structures

The results for the Medway Bridge, Mancunian Way, Western Avenue and Braidley Road Bridge are shown on Figures 9 to 12 which are reproduced in Appendix 11.13.

For these box girder bridges, strains in the top and bottom slabs were plotted as the mean of all the gauges distributed over the
width of each slab respectively.

The trend of the total strain curves is seen to be linear with the logarithm of time with superimposed cyclic seasonal movements of expansion in winter followed by contraction in the summer corresponding to the seasonal wetting and drying out of the concrete.

The effect of casting and prestressing a bridge in the winter is to slow down the development of creep and shrinkage, which then mainly takes place after the following spring to give a greater over-all movement.

(b) Observed results on small specimens

It has been found that small specimens kept outside are useful in predicting movements on the full-scale as the effect of climate is allowed for, although the movements in the specimens are large, unless the size effect is allowed for by banding the specimens with a waterproofing membrane around their centres, in such a way, that the depth of waterproofing was equal to the mean thickness of the box sections of the bridge (Tyler 12.16)*.

The creep in small specimens under load is shown up by the results for the River Air Bridge and Mancunian Way in Figures 19 to 20 which are reproduced in Appendix 11.13.

A conclusion from the results for several bridges was that specimens outside behave generally as those inside at a constant 20°C and 80% relative humidity, apart from the cyclic seasonal swings found with the outside specimens. Thus the English climate is roughly equivalent to constant 20°C and 80% RH from the point of view of creep and shrinkage in concrete.

(c) Summary of Results for Creep

(1) The creep factors obtained directly from the experimental results (i.e. therefore apparent creep factor $\phi_a$) are summarized in Figure 4.5.5. They were calculated for $t_0$, taken as 10 000 days, by scaling off appropriate ordinates for the total strain $\varepsilon_t$, and the shrinkage $S$, from the movement/log time curves.
Creep $\varepsilon_\infty$ is given at $t = \infty$ by
$$\varepsilon_\infty = \varepsilon_T - S_\infty - \varepsilon_0$$

Where $\varepsilon_0$ = Elastic strain from results on small specimens for the given prestressing force at the centroid of the cross section.

$\varepsilon_T$ = Total strain and $S$ = Shrinkage strain.

The apparent creep factor "$\varphi_a$" for reinforced concrete, loaded by a decaying prestressing force is given by $\frac{\varepsilon_\infty}{\varepsilon_0}$ and is a measure of creep at the centroid of a cross section since at the centroid the strain only depends on the P/A effect of the prestressing force. At any other level in the cross section, the strain also depends on the bending moments due to both external forces and redistribution of internal forces.

Therefore $\varphi_a = \frac{E}{\sigma} (\varepsilon_T - S_\infty) - 1$

where $\sigma$ is the prestress at the centroid of the cross section.

$E$ is the modulus of elasticity of the concrete.

Whilst there is a large scatter, the upper envelope of results in Figure 4.5.5. is seen to reduce from $\varphi_a = 2.2$ for prestressing soon after casting to 1.5 after about 100 days with a typical value of about 2 for prestressing from 7 to 14 days.

(2) Corresponding values for the creep factor $\varepsilon_\infty$ for plain concrete under constant load may be derived from the values of the apparent factor $\varphi_a$ by allowing for the decaying prestressing force and the effect of reinforcement as indicated in Tyler (12.1)*. The values of $\varphi_\infty$ are plotted in Figure 4.5.6.
It can be seen that the creep value $\phi_{\infty}$ for constant loading and unreinforced concrete is about 20% greater than $\phi_a$.

(3) The calculation of $\phi_{\infty}$ for the bridges enables creep on the full scale to be compared with that in small, unreinforced specimens under constant stress in creep rigs. It can be seen from Figure 4.5.6 that there is a scale effect, although it is not significant for design purposes.

(v) Evaluation of Observed Shrinkage Strains

(a) Comment

The shrinkage referred to is that primarily arising from moisture loss after the heat of hydration had dissipated, which corresponds roughly to the stripping of shutters ($\approx 120$ h in Britain).

Initially shrinkage was only measured on small specimens but later larger elements corresponding in thickness to the particular elements of the bridge itself were used.

(b) Medway Bridge

The mean shrinkage of four $711 \times 152 \times 152$ mm specimens, housed within the box beams of the bridge, as recorded by a Demec gauge is given in Figure 9 which is reproduced in Appendix 11.13. Projecting to $t = 10,000$ days the shrinkage in the companion specimens will be
approximately equal to the total movement in the bottom slab; \(420 \times 10^{-6}\) which is the sum of the elastic, creep and shrinkage movements in the slab. The actual shrinkage in the box beams themselves was calculated to be \(120 \times 10^{-6}\) (Tyler 12.12)*, thus showing a large size effect.

(c) Chiswick Cantilever

This was the largest cross section instrumented, being about 2.55 m deep by 2.75 m wide at the cantilever root. The total measured movement in the compression flange of the cantilever, inclusive of creep, shrinkage and elastic, was less than \(200 \times 10^{-6}\) in 5 years (Tyler 12.11)*, which is less than the value of shrinkage alone specified in CP 115 (12.52)*.

The shrinkage at \(t = 10,000\) days was estimated to be about \(50 \times 10^{-6}\), the low value being caused by the large size.

(d) Mancunian Way

For this structure, both \(508 \times 102\) mm specimens and a full scale reinforced precast unit were used for shrinkage estimation. The results of the movements of the full scale unit are shown in Figure 19, which is reproduced in Appendix 11.13 and show a net movement of \(23 \times 10^{-6}\) (Tyler 12.14)*.

The lack of movement in the shrinkage control unit is in contrast to the indicated shrinkage of the transverse gauges in the full structure (Figure 10 reproduced in Appendix 11.13) of \(S_\infty = 170 \times 10^{-6}\).

The probable explanation of the lack of movement in the control unit is that unlike in the actual structure, the top slab of the unit is not covered by a waterproofing membrane and the unit is stored in a forest glade and thus for long periods the concrete is not likely to dry out.

(e) Western Avenue

The effect of reinforcing a slab upon shrinkage is shown by a comparison of curves 1 and 2 of Figure 23 reproduced in Appendix 11.13 which show that after setting, the shrinkage in the reinforced slab (curve 1) is \(S_\infty = 120 \times 10^{-6}\) which is about three-quarter of that in the unreinforced one (curve 2) which is \(S_\infty = 170 \times 10^{-6}\). It is interesting
to note that a shrinkage was obtained in the fresh concrete which is probably due to the use of rapid hardening Portland Cement.

In contrast the shrinkage curve 4 (of figure 2 of Appendix 11.13) for an in-situ unreinforced concrete slab made with ordinary Portland Cement shows the usual initial expansion on setting to be followed by shrinkage to give $S^\infty = 100 \times 10^{-6}$.

(f) **London Bridge**

For London Bridge, an endeavour was made to correlate the shrinkage of small specimens with that on the full scale. Acoustic strain gauges were cast into one of the actual precast units and at the same time small specimens 508 x 102 x 102 mm in size were cast and waterproofed. Also cast was a slab of concrete 1,5 x 1,5 x 0,305 m in size, the 0,305 m being a typical thickness for the flange of the precast units.

The results for the actual precast unit, the 305 mm thick slab and a 508 x 102 x 102 specimen waterproofed with a 305 mm wide band all around are plotted in Figure 4.5.7 (Tyler 12.16a)*

![Figure 4.5.7](image)
The graphs show that while the shrinkage values from the waterproofed specimen agree well with those from the unit itself, the shrinkage values for the slab are about double. There is no explanation for this as the slab was reinforced the same as the flange element of the unit and exposure conditions were the same, although the slab was horizontal and the units vertical.

(g) **Summary of Results for Shrinkage**

The shrinkage values estimated for \( t = 10000 \) days are summarized in Figure 4.5.8 (Tyler 12.1)* in which a relationship between shrinkage and volume/surface area factor is indicated similar to that derived by Hansen and Mattock (12.40)*.

In general each plotted value represents the average shrinkage from a number of gauges buried in a structure which has a nominal amount of secondary reinforcement or for small unreinforced specimens the average of a batch.

![Figure 4.5.8](image-url)

**FIGURE 4.5.8 (TYLER 12.1)*

(vi) **Conclusions**

The main conclusions of this investigation are:

(a) The most likely cause of cracking in well cured concrete, within a few days after casting, is the early-age thermal movement set up by the hydration of cement as on the full scale, shrinkage due to moisture loss only occurs slowly

(b) The magnitude of shrinkage of bridge structures in Britain constructed to British Codes of Practice is likely to be of the order of 50 to 200 \( \times 10^{-6} \) at \( t = 10000 \) days, due to the mild and humid British climate.
(c) The creep factor, measured as the ratio of creep at \( t = 10000 \) days to elastic strain on prestressing is likely to be about \( 2.0 \), allowing for the effect of a decaying prestressing force.

4.5.5 New Zealand Investigation

(A) In-Situ Testing of Bridges

(i) Object

Since the early 1970's the Road Research Unit of the New Zealand National Roads Board has been co-ordinating NZ research into the structural behaviour of concrete bridges.

(ii) Structures Instrumented

Under the supervision of Priestley (12.5), a complex multi-cell prestressed concrete box deck forming part of the Bowen St Overpass was instrumented.

(iii) Instrumentation

Strains were monitored by vibrating-wire and demountable mechanical strain gauges. The vibrating wire strain gauges were cast into the concrete at two sections i.e. Section A-A at centre of span and Section B-B adjacent to interior support to measure longitudinal, transverse and shrinkage strains.

The shrinkage strains were monitored by gauges embedded in stress-free areas of the concrete, created by blockouts extending through the full thickness of web, deck or soffit slab. These blockouts were physically separated from the surrounding concrete on three sides by a layer of polystyrene, resulting in a separation from the stress field with identical curing conditions to the stressed concrete.

Gauges were also cast into large independent prisms formed from bridge concrete pours, simulating different areas of the cross section. Creep strains were assessed from these prisms using an "equivalent strain" technique.
(iv) Evaluation of Observed Strains

The concrete used had a specified W/C = 0.47 and a cement content of 380 kg/m³. At the time of prestressing the web and deck sections at Section A-A and Section B-B were 119 and 92 days old respectively, and prestressing was carried out over a period of 37 days. The prestressing was designed to balance the total dead weight and produce an uniform stress at Section A-A.

Figure 4.5.9 shows the total and shrinkage strain histories for the two instrumented sections, relative to a datum taken 24 hours after casting the webs and deck slab. Each line of Figures 4.5.9(a) and (b) represents the average values from all gauges at similar locations in the section. Section average strains in Figure 4.5.9(c) are the average value of all the gauges in each section.
The stress-induced strains can be obtained by subtracting the unrestrained shrinkage strains from the total strains, and the values of stress-induced strains occurring in various time intervals are given in Table 4.5.10.

The relatively large variation in the deck, web and soffit stress-induced strains occurring before prestressing result largely from locked-in thermal strains due to heat of hydration, since temperature measurements 24 hours after casting indicated a 20°C difference between the centre of the webs and the top and bottom surfaces. Cooling of the web was restrained by the deck and soffit slabs, thus inducing compressive strains in the deck and soffit, and tensile strains in the web.

Using the CEB-FIP Code (12.44)* the estimated creep strain in Section A-A at \( t = 10,000 \) days is \( 270 \times 10^{-6} \) and this compares with the average recorded value of \( 146 \times 10^{-6} \). The average shrinkage value from CEB-FIP for \( t = 10,000 \) days is \( 120 \times 10^{-6} \) which compares with an average recorded value of \( 340 \times 10^{-6} \). (Dr Priestley does not comment on this)

(v) Conclusion

The observed creep and shrinkage strain of a complex box-girder bridge differed significantly from those predicted by design methods.

(B) Creep and Shrinkage of a Bridge Building Concrete

(i) Object

The object of the investigation carried out by Bryant (12.4)* which also formed part of the NZ research co-ordinated by the Road Research Unit was to attempt to obtain creep and shrinkage values for a typical bridge building concrete that are relevant to the actual conditions at a bridge.

(ii) Experimental Investigation

Creep and shrinkage specimens were made on three separate occasions (mixes A, B and C) over a period of 7 months from concrete supplied by

---

**TABLE 4.5.10**

(PRIESTLEY 12.5)*

<table>
<thead>
<tr>
<th>Time Interval</th>
<th>Deck</th>
<th>Web</th>
<th>Soffit</th>
<th>Section Ave</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before stressing</td>
<td>47</td>
<td>-29</td>
<td>83</td>
<td>12</td>
</tr>
<tr>
<td>During stressing</td>
<td>313</td>
<td>383</td>
<td>437</td>
<td>408</td>
</tr>
<tr>
<td>1000 days after stressing</td>
<td>240</td>
<td>40</td>
<td>170</td>
<td>146</td>
</tr>
</tbody>
</table>

STRESS-INDUCED STRAINS SECTION AA

(\( \mu \text{m} / \mu \text{m} \) Tension = ve)
a ready-mix firm. Creep and shrinkage strains were monitored while the specimens were kept in 5 different places. Thus, creep and shrinkage strains recorded from specimens kept on top, inside and underneath the Grafton Gully Bridge No 1 can be compared to laboratory values obtained from specimens kept inside a constant temperature room and inside a fog room at the University of Auckland.

(iii) Evaluation of Observed Strain

(a) Results

The experiments are shown in Figure 3, reproduced in Appendix 11.14. On each graph the line through the solid circle is the average strain on the creep specimens and includes elastic, creep, shrinkage and temperature strains. The elastic strain (dashed line) was assumed to be constant at the value when the creep specimens were stressed and the line through the hollow circles is a summation of elastic strain plus strain recorded on shrinkage specimens which includes temperature strains.

In the graphs, tables and discussion the description shrinkage strains includes strains resulting from the temperature changes, although in an attempt to minimize extraneous temperature strains, the specimens at the bridge were read at dawn, before sunrise, during the period of stable daily temperatures.

(b) Effect of Season of Construction

Assuming that all 3 mixes have similar creep and shrinkage properties then the initial differences between the 3 mixes are due to the season of construction. After the specimens had all been 400 days on location there are no significant differences.

(c) Effect of Environment

At 400 days the fog room creep was about 50% of inside the bridge creep and the constant temperature creep was 115% of inside the bridge creep.

For shrinkage, the differences are even more with the fog room shrinkage at 400 days being -20% of inside the bridge shrinkage while the constant temperature shrinkage was 105% of inside the bridge shrinkage.
(d) Effect of Specimen Location at the Bridge

The graphs of Figure 4 reproduced in Appendix 11.14 are the averages of the experimental results for the 3 mixes thus eliminating any seasonal changes in weather and any difference between the graphs is due to location.

Specimens on and under the bridge have similar creep, but for the specimen inside the bridge the creep is 15% more. Shrinkage is more dependent on location with the inside shrinkage 100% more than on top and 25% more than under the bridge.

(iv) Prediction of Creep and Shrinkage Strains

In Table 4 reproduced in Appendix 11.14 predictions of creep and shrinkage based on CEB-FIP Code (12.44)* are given and the predictions are within 25% of the experimental ones.

(v) Conclusion

(a) The environment has a considerable effect on creep and shrinkage of specimens.

(b) Location of specimens at the bridge is relatively unimportant and differential creep and shrinkage stresses are mainly as a result of thick sections creeping and shrinking at a slower rate than thin sections.

(c) CEB-FIP Code (12.44)* can predict creep and shrinkage of specimens to within 25% of the experimental values.

NB It should be noted that the above investigations by Dr Byant concerned the behaviour of concrete specimens and NO correlation was done to see what the creep and shrinkage of an actual structure made with same concrete under the same conditions actually was.

4.5.6 Other Investigations

(i) Comment

A number of reports of investigations regarding the behaviour of
actual structures read were found to be only of indirect application although they bring out certain aspects and they will be referred to briefly.

(ii) Britain - "A study of some long-term strains measured in Two Sturctures" - Parrott (12.8)*

Introduction

This report represents strain measurements from prestressed ribbed slabs in a hospital prototype building and a similar reinforced ribbed slab in a telephone exchange neither which had yet been occupied.

Results and Discussion

The data presented lead to the following conclusions:

(a) Shrinkage of the site-stored concrete was size dependent. The apparent final value of shrinkage and the seasonal strain fluctuations both diminished with an increase in the effective section thickness of the concrete member.

(b) Prediction of average shrinkage (i.e. excluding seasonal fluctuations) and creep for outside exposure in the British Isles could conservatively be based upon a relative humidity of 80%, since the effective relative humidity for outside exposure is 80-90% (compared to the CEB recommended 'usual exterior' value of 70% for the continent.

(iii) Portugal - "Creep Effects in some Arch and Cantilever Bridges" - Borbes, Marecos and Trigo (12.23)*

Introduction

When measuring strains the main difficulty consists in distinguishing the effects of creep, shrinkage and temperature. In order to overcome this difficulty acoustic strain gauges were embedded in the structures at three different conditions i.e. (i) active gauges to measure total strains; (ii) compensating gauges introduced inside double wall cylinders to read shrinkage and temperature effects; (iii) controlled gauges also inside double wall cylinders but with metallic cushions filled with oil placed at the top of the boxes to allow known stresses to be applied on the cylinders for creep tests.
Results and Discussion

The results on creep obtained at an arch bridge and at a prestressed cantilever viaduct agree closely with the creep calculated by the expression presented in the CEB-FIP : 1970 Code. However, for another arch bridge the correlation between measured and computed creep is 100% out, which could be due to the fact that the theoretical coefficient $k_t$ (influence of time) takes a very low value due to the great thickness of the arch.

(iv) Japan - "Measurements of Creep and Shrinkage in Actual Prestressed Concrete Bridges". Kakubu, Goto, Ozaka, Okamura & Momoshimol (12.24)*

Introduction

In Japan, prestressed concrete bridges are designed in accordance with the Standard Specifications for Design and Construction of Prestressed Concrete of the Japan Society of Civil Engineers which was drawn up several years ago. In order to investigate the validity of the creep and shrinkage requirements of the code, measurements of time dependent deformations were carried out on 21 prestressed concrete bridges which represent almost all types of prestressed bridges in Japan.

Results and Discussion

Since neither the Japanese Code nor the climatic conditions etc. in Japan are generally known in South Africa, the actual observed measurements are of little interest although the following general observations made are:

(a) It is difficult to estimate the exact values of shrinkage and creep in concrete based on the measurements in actual prestressed concrete bridges. The main reason is that exact values of elastic strain of concrete in structural members are hardly obtainable and moreover, a separation between the values of shrinkage and creep requires that shrinkage be measured in the unstressed parts of the bridge.

(b) Shrinkage strain in concrete is affected by many factors. In fact, an example was found that the difference in 5 years shrinkage strains between each side of the cantilever of the cantilever prestressed concrete girder amounted to $150 \times 10^{-6}$. 

STATE OF THEORY
4.5.7 Conclusion

Thorough research has been carried out in Germany and Britain to correlate the theoretically predicted behaviour of concrete with actual observed behaviour of sophisticated prestressed concrete bridges and it has been found that for bridges in:

(a) In Germany the Theory and Practice agree well and the theoretical values based on the DIN 4227 and the CEB-FIP:1970 Codes are in general on the safe side.

(b) In Britain, due to the unique mild climate and constant high humidities the simple approach used in CP 115 has generally given satisfactory results as well.

(c) When using the DIN 4227 and the CEB-FIP : 1970 Codes, which were based on Continental Conditions allowance must be made for the milder and wetter British climate.
5. SITE INVESTIGATIONS

5.1 Introduction

Over the years, visual inspections of bridges in South West Africa had shown that in certain parts of the country, the deflection and movement of the decks was more than expected. It was therefore assumed that the aggregates used in the manufacture of the concrete must have very unfavourable creep and shrinkage characteristics.

In the early Seventies the decision was taken to go ahead with the construction of the Windhoek Western Link Road (Freeway) which included a number of Interchanges and river crossings. The configuration of the interchange structures and some of the river structures was such that multi span, prestressed, continuous, concrete decks were the most economical superstructure solution. For the design of prestressed concrete structures, an accurate prediction of the creep and shrinkage characteristics of the concrete is important. Although it was known that the concrete aggregates were suspected to have unfavourable creep and shrinkage characteristics no actual values were known. At this stage the laboratory investigation into the creep and shrinkage properties of the also suspect Port Elizabeth Concrete aggregates (12.6)*, had been completed and found to be very high. It was decided to use these values for the design of the Windhoek Western Link Road structures since it was felt that it was unlikely that the creep and shrinkage characteristics of the Windhoek concrete aggregates were more than those of the Port Elizabeth concrete aggregates.

However, some doubt was expressed as to whether the high creep and shrinkage values assumed for the Windhoek concrete aggregates were really justified and since considerable economic benefit could be derived by using lesser values for the design of future reinforced and prestressed concrete structures in the Windhoek area it was...
decided to conduct measurements of creep, shrinkage and elastic strains in a suitable structure constructed during the first stage of the Windhoek Western Link Road.

5.1.2 DESCRIPTION OF STRUCTURE

(a) General

The structure selected for instrumentation was the Brakwater South Interchange bridge. The bridge carries a district road over the Windhoek Western Link Road as shown on Sketch 5.1.1 and the superstructure consists of a 2 span, continuous prestressed concrete, two cell, shallow box deck as shown.

The bridge is prestressed in the longitudinal direction by means of 22 No draped cables each consisting of 12 No x 15,2 mm dia 7HI strands, placed in the webs. In the transverse direction only the centre cross beam over the single pier is prestressed by means 19 No curved cables, each consisting of 12 No x 15,2 mm dia 7HI strands. The CCL prestressing system was used for both the longitudinal and transverse prestressing. The cross beams at the abutments, the small mid-span cross beams as well as the top slab are reinforced concrete.

The abutments and central pier are founded on sound schist and therefore no settlement of the foundations can occur. Thus there is no possibility of additional bending moments being introduced into the deck due to settlement of foundations.
(b) **Concrete Aggregates**

**Description**

The concrete aggregates available commercially in the Windhoek area are obtained from the Klein Windhoek River by crushing and screening the alluvial deposits. These deposits consist mainly of quartzitic particles which originate from mica schist formations interbedded with quartzitic bands. The particle sizes vary between 0.05 mm and 1000 mm and more, and the size distribution varies considerably even in the same deposit.

The Windhoek aggregates have been investigated by Müller (12.63)* who found that particles larger than 3 mm consisted almost exclusively of quartzite while particles smaller than 3 mm consisted predominantly of quartzite with up to ten percent by weight of biotite and only traces of muscovite, feldspar and garnet. The investigation into the effect of the mica contained in the fine aggregate upon the workability, compressive strength and drying shrinkage of concrete made with such aggregates found that the effects depended much more on the nature of the micaceous particles than on their quantity. The conclusions drawn from the investigation were that:

(i) the workability, expressed in terms of slump, decreases as the mica content increases

(ii) The compressive strength of concrete decreases as the mica content increases. With biotite the decrease in strength was small, but significant for muscovite.

(iii) The presence of mica tends to increase shrinkage, though not at an alarming rate and without conspicuous difference between the effect of biotite and that of muscovite.
Fine Aggregate

This consists of river sand obtained by washing the crushed and screened quartz alluvial deposits and has the following grading analysis:

<table>
<thead>
<tr>
<th>SCREEN ANALYSIS</th>
<th>SABS Screens</th>
<th>% Passing by mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>4750 μm</td>
<td>96</td>
<td></td>
</tr>
<tr>
<td>2360 μm</td>
<td>83</td>
<td></td>
</tr>
<tr>
<td>1180 μm</td>
<td>68</td>
<td></td>
</tr>
<tr>
<td>600 μm</td>
<td>52</td>
<td></td>
</tr>
<tr>
<td>300 μm</td>
<td>23</td>
<td></td>
</tr>
<tr>
<td>150 μm</td>
<td>9</td>
<td></td>
</tr>
</tbody>
</table>

FINENESS MODULUS = 2.72

Coarse Aggregate

This consists of crushed and screened quartz alluvial deposits and has the following grading analysis:

<table>
<thead>
<tr>
<th>SCREEN ANALYSIS</th>
<th>SABS Screens</th>
<th>% Passing by mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>37.5 mm</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>26.5 mm</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>19.00 mm</td>
<td>83</td>
<td></td>
</tr>
<tr>
<td>13.20 mm</td>
<td>23</td>
<td></td>
</tr>
<tr>
<td>9.50 mm</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>4.75 mm</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>
(c) Concrete Mix Design

Initially, in view of the suspect creep and shrinkage characteristics of the aggregates mainly due to the presence of mica, it had been specified that no admixtures were allowed. However, this requirement was later relaxed, in view of the difficulties experienced by the contractor in placing the concrete.

The following Mix 40 MPa, which uses the Pozzolith 100 XR Retarder has been designed to have the same strength and slump as the mix without an admixture (see Section 4.1.5)

<table>
<thead>
<tr>
<th>CONSTITUENTS</th>
<th>Wt. PER m³ OF CONCRETE</th>
<th>COMMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement OPC</td>
<td>445 kg</td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>515 kg</td>
<td>Surface dried which include 2½% moisture</td>
</tr>
<tr>
<td>13,2 mm stone</td>
<td>420 kg</td>
<td>Surface Dried</td>
</tr>
<tr>
<td>19,0 mm stone</td>
<td>860 kg</td>
<td>Surface Dried</td>
</tr>
<tr>
<td>Water</td>
<td>200 litre</td>
<td></td>
</tr>
<tr>
<td>Additive (Pozzolith 100XR)</td>
<td>400 ml</td>
<td></td>
</tr>
</tbody>
</table>

DESIGN SLUMP = ± 50 mm

NB

The cement contents of the mix is on the high side but it had been found that the cement strength could drop without warning by 10% while still complying with SABS 471.
(d) **Concreting Sequence of Superstructure**

The two span superstructure was designed to be constructed in one operation, and although the contractor erected the scaffolding, shuttering and placed the reinforcing steel and prestressing ducts for the two decks together, it was obviously not possible to concrete the superstructure in one operation.

In order to limit differential shrinkage stresses it had been specified that

(i) the bottom slab and webs of the boxes had to be cast in one operation and

(ii) that the top slab then had to be cast within 7 days.

The contractor elected to cast the two spans in six stages as shown in sketch 5.1.2 which also gives the dates of casting as well as the average 28 day cube strength of the concrete in the top and bottom slabs.

![Sketch 5.1.2](image)

**SKETCH 5.1.2**

**ELEVATION OF BRIDGE SHOWING CONSTRUCTION STAGES (NOT TO SCALE)**

**EXPERIMENTAL INVESTIGATION**
The average 28 day cube strength of the concrete

(i) in the mid span cross beam where the shrinkage strain is measured and

(ii) in the centre web where the total strain is measured

are as shown in sketch 5.1.3:

---

**SKETCH 5.1.3**

**ELEVATION OF ONE SPAN SHOWING CENTRE WEB CONCRETE STRENGTH**

*(NOT ON SCALE)*

---

**Prestressing Sequence of Superstructure**

The loss of prestress due to creep of concrete is very dependent on the age at which load is applied to the concrete and it was therefore specified that prestressing could only commence 14 days after the temporary access holes left in the top slabs to remove the interior top slab soffit shutters had been concreted and this concrete had reached a strength of 35 MPa.
In order to limit the stresses in the centre cross beam over the single pier, the following prestressing sequence was specified:

(i) Stress 50% of centre cross beam cables - this was done on 6.11.78

(ii) Stress all the longitudinal cables - this was commenced on 6.11.78 and completed on 11.11.78

(iii) Stress remainder of centre cross beam cables - this was done on 11.11.78

(f) Observation
No unusual behaviour was noticed during the construction of this bridge and to date the bridge has performed satisfactorily.

5.1.3 Instrumentation

(a) Measurements required
The prime object of the site investigation was to observe the actual creep, shrinkage and elastic strains in the superstructure, but these are influenced by extrinsic factors such as temperature and relative humidity, which also have to be measured.
It was decided that the following measurements would be recorded:

(i) Total strain  
(ii) Shrinkage strain  
(iii) Temperature of air inside and outside of box deck  
(iv) Relative humidity of air inside and outside of box deck.

(b) Total Strain

It is not possible to observe creep strains without also observing shrinkage strains (and possibly temperature strains).

As discussed in Section 4.4.4 the strains had to be measured by means of a Demec demountable gauge and it was decided to take total strain readings on the centre web of one span at the following points, shown on sketch 5.1.4:

At each measuring point, four targets were fixed to the concrete by means of epoxy adhesive, so that the calculated strain at each point would be the average of three readings.
(c) As discussed in Section 4.4.4., the strains had to be measured by means of a Demec demountable gauge and it was decided to take shrinkage strain measurements on the relatively unstressed mid span cross beams, at the points shown on Sketch 5.1.5.

At each of the two measuring points, four targets were fixed to the concrete by means of epoxy adhesive.

(d) Temperature of Air

The temperature inside the box was recorded on a thermometer left permanently in the box; while the temperature outside the box was recorded by means of a thermometer kept outside at the nearby site offices.

(e) Relative Humidity

The relative humidity inside the box was recorded continuously for the first three weeks but in view of the difficulty of access to the inside of the box deck, these readings were then discontinued.

The relative humidity outside the box was recorded by means of a wet and dry bulb thermometer kept outside at the nearby site offices.
Field Observations

(a) Time of day when readings should be taken

As discussed in Section 4.3.4, all parts of a concrete bridge are at about the same temperature early in the morning after sunrise before the sun has warmed up the bridge and the strain readings were therefore taken between 6 am and 8 am depending on the time of the year.

(b) Total Strain

The object of these readings is to measure the elastic shortening strain of the concrete as the longitudinal prestressing force is applied and the subsequent creep strain of the concrete due to the longitudinal prestressing and dead load stresses.

The first reading was taken in the morning before the longitudinal prestressing force was applied and the next reading in the morning after the longitudinal prestressing force had been applied. From then on readings were taken once a day for the first 10 days (except Sundays) and then about once every 10 days for the next 90 days except for weekends and then about once every 100 days for the next 900 days.
(c) **Shrinkage Strain**

As discussed in Section 4.1.2 the shrinkage to be measured is the one that occurs once the heat of hydration has dissipated and the first readings were therefore taken 72 hours after the cross beam had been cast, even before the top slab had been concreted. The next readings were then taken once a day for the next 10 days and then about once every 10 days until the total strain readings commenced. From then on the Shrinkage strain readings were taken at the same time as the total strain readings.

The observed shrinkage strain readings are recorded in Appendix 11.3(b).

(d) **Temperature of Air**

The air temperature readings inside and outside the box were taken at the same time as the total strain and shrinkage strain readings were taken.

The observed air temperature readings are recorded in Appendix 11.3(a).

(e) **Relative Humidity**

The relative humidity readings were taken at the same time as the total strain and shrinkage strain readings were taken. The observed relative humidity readings are recorded in Appendix 11.3(a).
5.1.5 Analysis of Field Observations

(a) Total Strain of Concrete

(i) General
As discussed in Section 4.5.4(iv)(c), it is best to evaluate the creep strain at the Neutral Axis, since at the Neutral Axis the creep strain only depends on the direct \( \frac{P}{A} \) effect of the prestressing force. Therefore the gauge point "B" readings will be used to evaluate the creep strain.

(ii) Reduction of Readings
At each stage, the change in total strain i.e. creep strain, shrinkage strain and axial temperature strain relative to the reading observed when longitudinal prestressing was completed on the 11.11.78 has been calculated, and this value then averaged for the three values observed to give the "mean change in total strain between these two readings" as calculated in Appendix 11.3(a).

(b) Shrinkage Strain

(i) General
The shrinkage strain readings also include the axial temperature strain, which have to be allowed for when determining actual shrinkage strain.

(ii) Shrinkage Strain Component of Total Strain
Both the observed total strain and shrinkage strain readings include the same axial temperature strain, which is automatically eliminated when the shrinkage strain reading is deducted from the total strain reading to give the creep strain.

At each stage the change in shrinkage strain relative to the reading observed when the longitudinal...
prestressing was completed on the 11.11.78 has been calculated and this value then averaged for the six values observed to give the "mean change in shrinkage strain between these two readings" as calculated in Appendix 11.3(b).

(iii) True Shrinkage Strain

As already mentioned the observed shrinkage strain readings also include an axial temperature strain component which must be eliminated by correcting all readings to the air temperature observed when the readings commenced as discussed in Section 4.3.5(iv) using a coefficient of thermal expansion of 12 x 10^{-6} per °C. At each stage, the change in shrinkage and temperature strain relative to the readings observed when measurements commenced on 28.9.78 has been calculated, and this value then averaged for the six values observed and then corrected for the axial temperature strain due to the difference between the air temperature at each stage to the 14°C air temperature observed when measurements commenced on 28.9.78 as calculated in Appendix 11.3(d). (The readings when the air temperature was below about 10°C have not been considered as the mean bridge temperature is unlikely to fall so low).

In Appendix 11.3(g) two graphs of true shrinkage strain up to 1 000 days have been plotted, obtained by basing the temperature correction on

(i) outside air temperature at reading, and
(ii) the air temperature inside box at reading.
The estimated shrinkage strain at \( t = 10\,000 \) days can only be obtained by projecting the \( t = 1000 \) day values and this has been done by the following methods:

**Method 1**

This is the straight line Method used by Tyler (12.1)* and gives a shrinkage strain from \( t = 3 \) to \( t = 10\,000 \) days of \( 48 \times 10^{-5} \) when the temperature corrections is based on outside air temperature and \( 38 \times 10^{-5} \) when the temperature correction is based on temperature of air inside box.

**Method 2**

This is based on coefficient \( k_e \) as given in diagram (6)-R12.31 of the CEB-FIP (1970) Recommendations (12.44)*, which depends on the theoretical thickness \( e_m \) of the member, in this case the midspan cross-beam ie

\[
e_m = \frac{\text{area of section}}{0.5 \text{ perimeter in contact with atmosphere}}
\]

\[
e_m = \frac{119.0 \times 30.5}{0.5 \times 2(103.0 + 30.5)} = 27 \text{ cm}
\]

\( \therefore \) from diag (6)-R12.31 for \( e_m = 27 \text{ cm} \) \( k_e \) at 1000 days = 0.8

Using this value of \( k_e \), the shrinkage strain from \( t = 3 \) to \( t = 10\,000 \) days is given as \( 43 \times 10^{-5} \) when the temperature correction is based on outside air temperature and \( 33 \times 10^{-5} \) when the temperature correction is based on temperature of air inside box.

(c) **Elastic Strain**

As the on-site prestressing operations took 4 days to complete the change in strain observed at the various points on the centre web as a result of the application of the longitudinal prestress must also include some creep as the "apparent" Modulus of Elasticity obtained calculated in the table below is too low.
The laboratory tests reported on in Section 5.2.5(b) showed that the Modulus of Elasticity of concrete cylinders made with similar concrete was $31.4 \times 10^3$ MPa at 28 days, and assuming that the Modulus of Elasticity of the bridge concrete is similar then the creep strain component of the observed strain due to the application of the longitudinal prestressing force is

<table>
<thead>
<tr>
<th>SECTION</th>
<th>0.1L</th>
<th>0.4L</th>
<th>0.75L</th>
<th>0.9L</th>
</tr>
</thead>
<tbody>
<tr>
<td>Observed Strain (mm/mm)</td>
<td>$-22.7 \times 10^{-5}$</td>
<td>$-25.5 \times 10^{-5}$</td>
<td>$-18.7 \times 10^{-5}$</td>
<td>$-18.7 \times 10^{-5}$</td>
</tr>
<tr>
<td>Initial Prestress force after lock-off kN</td>
<td>38 800</td>
<td>40 600</td>
<td>40 200</td>
<td>39 200</td>
</tr>
<tr>
<td>Area of Section mm$^2$</td>
<td>$7.788 \times 10^6$</td>
<td>$7.788 \times 10^6$</td>
<td>$7.788 \times 10^6$</td>
<td>$7.788 \times 10^6$</td>
</tr>
<tr>
<td>&quot;Apparent&quot; Modulus of elasticity kN/mm$^2$</td>
<td>22.0</td>
<td>20.4</td>
<td>27.6</td>
<td>26.9</td>
</tr>
</tbody>
</table>

The above creep strains from $t = 0$ to $t = 3$ days after loading must be added to creep strains from $t = 3$ to $t = 10000$ days determined in Section 5.1.5(d).
(d) Creep Strain

As already noted the creep strain is obtained by deducting the shrinkage strain reading from the total strain reading.

At each stage, the "mean change in shrinkage strain" relative to the readings observed when longitudinal prestressing was completed on the 11-11-78 as calculated in Appendix 11.3(b) has been deducted from the "mean change in total strain" relative to the readings observed when longitudinal prestressing was completed on the 11-11-78 as calculated in Appendix 11(a) to give the "mean change in creep strain" for a decaying prestressing force as shown in Appendix 11.3(c). The "mean change in creep strain" has been plotted graphically to a logarithmic scale in Appendix 11.3(f) up to $t = 10000$ days.

The estimated creep strain at $t = 10000$ days can only be obtained by projecting the $t = 10000$ day values and the method used is based on a coefficient $k_e$ as given in diagram (6) - R12.31 of the CEB-FIP (1970) Recommendations (12.44)*, which depends on the theoretical thickness $e_m$ of the member, in this case the centre web of one span is

$$e_m = \frac{\text{area of section}}{0.5 \ \text{perimeter in contact with atmosphere}}$$

$$= \frac{165.0 \times 64.0}{0.5 \times 2(165 + 64)} = 46 \ cm$$

\[ \therefore \text{from diag (6)-R12.31 for } e_m = 46 \ cm \ k_e \text{ at } 10000 \text{ days } = 0.7 \]

the creep strain from $t = 3$ days to $t = 10000$ days for a decaying prestress force is as follows:

<table>
<thead>
<tr>
<th>SECTION</th>
<th>0,1L</th>
<th>0,4L</th>
<th>0,75L</th>
<th>0,9L</th>
</tr>
</thead>
<tbody>
<tr>
<td>Creep strain $t = 3$ to $t = 10000$ days from Appendix 11.3(f)</td>
<td>$-20 \times 10^{-5}$</td>
<td>$-24 \times 10^{-5}$</td>
<td>$-24 \times 10^{-5}$</td>
<td>$-18.5 \times 10^{-5}$</td>
</tr>
<tr>
<td>Initial Creep strain from Section 5.1.5(c)</td>
<td>$-6.8 \times 10^{-5}$</td>
<td>$-8.9 \times 10^{-5}$</td>
<td>$-2.3 \times 10^{-5}$</td>
<td>$-2.7 \times 10^{-5}$</td>
</tr>
<tr>
<td>$t=0$ to $t=3$ days</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total Creep strain $t=0$ to $t=10000$ days</td>
<td>$-27 \times 10^{-5}$</td>
<td>$-33 \times 10^{-5}$</td>
<td>$-27 \times 10^{-5}$</td>
<td>$-21 \times 10^{-5}$</td>
</tr>
</tbody>
</table>

EXPERIMENTAL INVESTIGATIONS
The apparent Specific creep for the bridge, which is a measure of the creep at the centroid of a concrete member reinforced with secondary steel for a decaying prestressing force is defined as

\[
\text{Specific creep} = \frac{\text{Total creep strain (from above table)}}{\text{Initial Prestress force/Area of Section (from table in Section 5.1.5(c))}}
\]

<table>
<thead>
<tr>
<th>SECTION</th>
<th>0.1L</th>
<th>0.4L</th>
<th>0.75L</th>
<th>0.9L</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total creep strain</td>
<td>(-27\times10^{-5})</td>
<td>(-33\times10^{-5})</td>
<td>(-27\times10^{-5})</td>
<td>(-21\times10^{-5})</td>
</tr>
<tr>
<td>Initial Prestress force/Area of section</td>
<td>5.0 MPa</td>
<td>5.2 MPa</td>
<td>5.2 MPa</td>
<td>5.0 MPa</td>
</tr>
<tr>
<td>Apparent Specific Creep</td>
<td>(54\times10^{-6}/\text{MPa})</td>
<td>(64\times10^{-6}/\text{MPa})</td>
<td>(52\times10^{-6}/\text{MPa})</td>
<td>(42\times10^{-6}/\text{MPa})</td>
</tr>
</tbody>
</table>

Based on the above results, the estimated average specific creep of the concrete from \(t=0\) to \(t=10,000\) days for a decaying prestressing force is \(56 \times 10^{-6}/\text{MPa}\).

**NB** It must be remembered that this value is for concrete loaded at 40 days after casting, compared to the usual 14 days after casting. Using coefficient \(k_d\) from diagram (3) - R12.31 of the CEB-FIP (1970) Recommendations (12.44)* the above estimated average specific creep for concrete loaded at 14 days after casting becomes \(56\times10^{-6} \times (\frac{1.2}{0.9}) = 75\times10^{-6}/\text{MPa}\)

### 5.1.6 Summary and Comments

**a) Elastic strain**

As the on-site prestressing operations took 4 days to complete, the change in strain observed after 4 days also included some creep strain, and the actual Modulus of Elasticity of the bridge concrete cannot be calculated.
For the purpose of these investigations the Modulus of Elasticity of the bridge concrete at stressing when the concrete was 40 days old has been assumed to the same as the Modulus of Elasticity of Concrete Cylinders tested in the laboratory at 28 days ie 31.4 kN/mm²

(b) True shrinkage strain
As discussed in Section 5.1.5(c)(iii) the estimated shrinkage strain at \( t = 10000 \) days depends on the assumed temperature correction and method of projecting the \( t=1000 \) day values. It is considered that a good approximation is obtained if the temperature correction is obtained from the mean of the outside and inside temperatures, and the CEB-FIP (1970) Method is used to project the \( t=1000 \) days values to \( t=10000 \) days.

Therefore, estimated shrinkage strain \( t=3 \) to \( t=10000 \) days = \( 38 \times 10^{-5} \).

This value is somewhat lower than had been expected for concrete made with Windhoek aggregates, based on laboratory tests on small specimens.

(c) Creep Strain
The apparent specific creep for the bridge based on a decaying prestressing force, applied to the concrete 40 days after casting from \( t=0 \) to \( t=10000 \) days was found to be \( 56 \times 10^{-6} /\text{MPa} \) (If the concrete had been loaded at 14 days after casting, then this specific creep would have increased to \( 75 \times 10^{-6} /\text{MPa} \))
LABORATORY INVESTIGATIONS

5.2.1 Introduction

The design of sophisticated concrete structures such as prestressed concrete or arch bridges requires a knowledge of the characteristics of the actual concrete to be used in the structures.

In the laboratory it is relatively easy to determine under controlled conditions the following properties of concrete:

(i) compressive strength
(ii) modulus of elasticity
(iii) specific creep plus shrinkage strain
(iv) coefficient of thermal expansion
(v) shrinkage of concrete to SABS

As the Laboratory work part of this investigation, the above properties of concrete were determined in the Laboratory of the Department of Civil Engineering of the University of Cape Town on test specimens. The concrete for these specimens was produced in the laboratory using samples of the actual cement, stone, sand and additive sent from Windhoek and mixed in the Mix proportions used on site.

Observation

It is interesting to note that the N.B.R.I. of the C.S.I.R. who as referred to in Section 3.3 are conducting a long term investigation into the movement and deflection of bridges in South West Africa actually made their laboratory specimens on site, obtaining concrete from the Ready Mixed Trucks as the Brakwater South deck was being cast. These specimens were then initially cured in Windhoek before being sent to Pretoria and subjected to creep, shrinkage etc. testing. This method will make the correlation between the behaviour of specimens and the actual structure more correct since it eliminates the laboratory effect introduced when the small quantities of concrete required for the casting of specimens are mixed in the laboratory.
However in day to day design practice one requires to know the characteristics of the concrete before a structure can be constructed and must therefore resort to mixing small quantities of concrete in a laboratory and hence this procedure was followed for this investigation.

5.2.2 Description of Methods of Testing

(a) General
Due to the fact that there is no single code available that covers the determination of all the properties of concrete that are required, resort has to be made to a number of codes i.e.

(b) Compressive Strength of Test Cubes
Making and Curing of Specimens
Six test cubes were made and cured in accordance with BS 1881: Part 3 (12.50a)* as follows:

(i) size of cubes = 150 mm
(ii) Compaction of test cubes = moulds filled in layers approximately 50 mm deep and each layer compacted by vibration by means of a vibrating table until the specified condition was attained.
(iii) curing = moulds covered for first 24 hours with damp sacks and moulds then stripped and cubes placed in water maintained at 20°C ± 1°C and kept there until just before the test.

Test for the compressive strength of cubes
The test cubes were tested in accordance with BS 1881: Part 4 (12.50b)* as follows:
(i) **Testing machine** - suitable capacity and capable of applying the load at the rate specified.

(ii) **Placing of cube in the testing machine** - test cubes placed in the machine in such a manner that the load is not applied to the top and bottom faces as cast.

(iii) **Loading** - the load applied without shock and increased continuously at a rate of 15 MN/m² per min (375 kN for 150 mm cubes) until no greater load can be sustained.

(iv) **Calculation** - compressive strength of each cube = maximum load sustained / Nominal cross-sectional area of cube

(c) **Modulus of Elasticity (Static) of Test Cylinders**

**Making and Curing of Specimens**

Two test cylinders were made and cured in accordance with BS 1881 : Part 3 (12.50(a))* as follows:

(i) **Size of cylinders** - 152.4 mm (6 ins) diameter by 304.8 mm (12 ins) long

(ii) **Compaction of test cylinders** - the moulds filled in layers approximately 50 mm deep and each layer compacted by vibration by means of a vibrating table until the specified condition was attained.

(iii) **End preparation of the concrete cylinders** - the moulds filled to within 6 mm of the top and then capped with a sand/cement mortar of the same w/c ratio as the concrete. Then a capping plate was pressed down and left in position until the mould was stripped.

(iv) **Curing** - moulds covered for first 24 hours with damp sacks and moulds then stripped and cylinders placed in water maintained at 20°C ± 1° and kept there just before the test.

**Test for the Static Modulus of Elasticity by means of an Extensometer**

The two test cylinders were tested in accordance with BS 1881 : Part 5 (12.50(c))*
(i) **Testing Machine** - suitable capacity and capable of applying the load at the rate specified and also of maintaining the load at any desired value.

(ii) **Strain Measurement** - Two sets of Demec gauge points required on opposite sides of each specimen and parallel to its axis, in such a way that the gauge points are symmetrical about the middle of the specimen and not nearer to either end of the specimen than a distance equal to half its diameter, (thus resulting in a gauge length of 150 mm for the size of cylinder used).

(iii) **Preliminary Loading** - First Loading - the load increased continuously at a rate of 15 MN/m² per min. until the average stress of \((C+2)\) MPa is reached (where \(C\) is one third of the average compressive strength of a set of 3 cubes of similar age tested just beforehand). The load then reduced to 1 MPa and strain gauge readings taken.

Second Loading - the load applied a second time to a stress of \((C+1)\) MPa and strain gauge readings taken before reducing the load to 1 MPa and again taking strain gauge readings.

(iv) **Loading** - The load then applied a third time at the same rate and strain gauge readings taken at ten approximately equal increments up to a stress of \((C+1)\) MPa. The average strain observed on the second and third loadings must not differ by more than 5%.

(v) **Calculation** - For the last cycle the strain vs stress graph for each of the two gauge points must be plotted and the best straight line drawn through each set of readings. The slope of these two lines must be determined and the average value found. The difference between the individual values for slope must be less than 15% of the average value, and the average value is then the modulus of elasticity of concrete.
Specific creep plus shrinkage strain of test cylinders

Making of specimens

Four test cylinders were made in accordance with BS 1881: Part 3 Section 5 (12.50(a)) as follows:

(i) **Size of cylinders** - 104.5 mm (4 ins) dia by 304.8 (12 ins) long as the creep rig available did not allow the use of 152.4 mm (6 ins) dia cylinders as specified by the Code.

(ii) **Compaction of test cylinders** - the moulds were filled in layers approximately 50 mm deep and each layer compacted by vibration by means of a vibrating table until the specified condition was attained.

(iii) **End preparation of the concrete cylinders** - the moulds filled to within 6 mm of the top and then capped with a sand/cement mortar of the same w/c ratio as the concrete. Then a capping plate pressed down and left in position until the mould was stripped.

Curing of Specimens

Four test cylinders were cured in accordance with ASTM C512-66T (12.48)* as follows.

(i) **Stripping of moulds** - The moulds were stripped after 24 hours.

(ii) **Moist curing** - The specimens then covered with wet sacks and placed in 100% Relative Humidity curing room at 23°C until the age of 7 days.

(iii) **Constant Humidity and Temperature room** - After the completion of the 7 days moist curing the specimens stored at constant Relative Humidity of 50% ± 4%, and constant temperature of 23°C ± 1°C until completion of test.

Test for Creep of Concrete

The two creep test cylinders and the two shrinkage strain control cylinders were tested in accordance with ASTM C512-66T (12.48)* as follows: (N.B. The original ASTM C512-66T and the new ASTM C512-76 are basically the same).
(e) Shrinkage of Concrete to SABS

Making and curing of specimens

Three test specimens were made and cured in accordance with SABS 718-1962 (12.51)* as follows:

(i) **Size of prisms** - 75 mm x 75 mm x 200 mm (nominally)

(ii) **Compaction of test prisms** - the moulds filled in layers approximately 25 mm deep and each layer compacted by vibration by means of a vibrating table until the specified condition was attained.

(iii) **Stripping of moulds** - cured for 24 hours under damp sacks and moulds then stripped.

(iv) **End preparation of prisms** - After stripping of moulds, 6 mm dia steel balls fixed with epoxy glue to the ends of the prisms and coated with grease to prevent corrosion.

(v) **Initial Curing** - after stripping prisms cured for further 48 hours under water.

(vi) **Final Curing** - After taking initial readings after 48 hours of initial water curing, the specimens stored at constant Relative Humidity of 50% ± 10% and constant temperature of 23°C ± 1°C until the second set of readings taken at 28 days after casting.

Testing for shrinkage

Three prisms were tested in accordance with SABS 718-1962 (12.51)* as follows:

(i) **Measuring device** - Dial gauge mounted in a rigid measuring frame, with a seating at the bottom end to locate the 6 mm dia steel ball epoxied to the end of the prism. An invar steel rod used as a standard length against which the readings of the gauge can be tested.
(ii) Measurements - Specimens removed from water at 72 hours (3 days) after casting and the distance between the steel balls epoxied to the ends of the prisms measured.

Specimens then stored in constant Relative Humidity and temperature room and at 28 days after casting the distance between the steel balls again measured.

(iii) Calculation - The shrinkage of concrete to S.A.B.S. is defined as the difference between the readings at 3 and 28 days, expressed as a percentage of the original distance.

5.2.3 Instrumentation and Laboratory Testing

(a) General

All the instrumentation and specialised equipment required to carry out the creep and shrinkage test etc. for this investigation had been assembled and tested previously by Mr Mackenzie of the Department of Civil Engineering, U.C.T. in his research into the creep and shrinkage of concrete.

Since the provision and development of equipment did not form part of this investigation, the equipment used will only be briefly referred to for clarity and amplified by means of suitable photographs where necessary.

(b) Concrete Mix

As outlined in clause 5.2.1 concrete for the samples was produced in the laboratory.
The mix proportions for the Mix 40 MPa as used for the Brakwater South Interchange are as follows:

<table>
<thead>
<tr>
<th>Constituents</th>
<th>wht per m³ of concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>445 kg</td>
</tr>
<tr>
<td>Sand</td>
<td>515 kg (surface dried which includes 2½% moisture)</td>
</tr>
<tr>
<td>13,2 mm stone</td>
<td>420 kg (surface dried)</td>
</tr>
<tr>
<td>19,0 mm stone</td>
<td>860 kg (surface dried)</td>
</tr>
<tr>
<td>Water</td>
<td>200 l</td>
</tr>
<tr>
<td>Additive (100 x R)</td>
<td>400 ml</td>
</tr>
<tr>
<td>Pozzolith</td>
<td></td>
</tr>
</tbody>
</table>

In the laboratory a volume of concrete had to be produced to allow the following specimens be cast from one batch.

<table>
<thead>
<tr>
<th>SPECIMEN DESCRIPTION</th>
<th>NO OFF REQUIRED</th>
<th>VOLUME m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength</td>
<td></td>
<td></td>
</tr>
<tr>
<td>150 x 150 mm cubes</td>
<td>6</td>
<td>0.021</td>
</tr>
<tr>
<td>Modulus of Elasticity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>152.4 mm φ x 304.8 mm long cylinders</td>
<td>2</td>
<td>0.011</td>
</tr>
<tr>
<td>Creep and shrinkage</td>
<td></td>
<td></td>
</tr>
<tr>
<td>104.5 mm φ x 304.8 mm long cylinders</td>
<td>6</td>
<td>0.015</td>
</tr>
<tr>
<td>Shrinkage prisms</td>
<td></td>
<td></td>
</tr>
<tr>
<td>75 x 75 mm square by 290 mm long prism</td>
<td>3</td>
<td>0.003</td>
</tr>
</tbody>
</table>

| NETT VOLUME OF CONCRETE REQUIRED | 0.050 |

In order to produce a volume of concrete of this order it was decided to use \( \frac{1}{15.4} \) of the quantities required for 1 cub metre to give about 0.05 m³ of concrete, (the slight discrepancies in the weights of sand and water occurred due to variations in the actual moisture content of the sand)
<table>
<thead>
<tr>
<th>Constituents</th>
<th>Wt used in Lab kg</th>
<th>Theoretical Lab Mix/ 445 kg cement kg</th>
<th>Actual Site Mix/ 445 kg cement kg</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>28.95</td>
<td>445</td>
<td>445</td>
</tr>
<tr>
<td>Sand</td>
<td>32.14</td>
<td>494.1</td>
<td>515 (4% too little sand actually used)</td>
</tr>
<tr>
<td>(2½% moisture)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13.2 mm stone</td>
<td>27.30</td>
<td>419.7</td>
<td>420</td>
</tr>
<tr>
<td>19.0 mm stone</td>
<td>55.90</td>
<td>859.3</td>
<td>860</td>
</tr>
<tr>
<td>Water</td>
<td>12.94</td>
<td>198.91</td>
<td>200 (0.5% too little water actually used)</td>
</tr>
<tr>
<td>Additive</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(100 x R Pozzolith)</td>
<td>0.026</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>W/C</td>
<td>0.45</td>
<td></td>
<td>0.45</td>
</tr>
<tr>
<td>Slump</td>
<td>+55 mm</td>
<td></td>
<td>+ 50 mm</td>
</tr>
</tbody>
</table>

(c) Compressive strength of Test Cubes

*Instrumentation* - electrically operated cube crushing machine with automatic load recording device, See Fig. 5.2.1. for crushed specimens.

Fig. 5.2.1
Laboratory Testing
(For making, curing and testing procedure see clause 5.2.2(b)
(i) 6 No 150 x 150 mm cubes cast on 17.10.78
(ii) 3 No cubes crushed on 31.10.78 (ie 14 days after casting) and the following results obtained.

<table>
<thead>
<tr>
<th>CUBE REF. NO</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. Load</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sustained kN</td>
<td>986</td>
<td>940</td>
<td>984</td>
</tr>
<tr>
<td>:. Compressive Strength (MPa)</td>
<td>43.8</td>
<td>41.8</td>
<td>43.7</td>
</tr>
</tbody>
</table>

(iii) 3 No cubes crushed on 14-11-78 (ie 28 days after casting) and the following results obtained.

<table>
<thead>
<tr>
<th>CUBE REF. NO</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. Load</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sustained kN</td>
<td>1115</td>
<td>1077</td>
<td>1119</td>
</tr>
<tr>
<td>:. Compressive Strength (MPa)</td>
<td>49.6</td>
<td>47.9</td>
<td>49.7</td>
</tr>
</tbody>
</table>

(d) Modulus of Elasticity (Static) of Test Cylinders

Instrumentation: electrically operated cube crushing machine with automatic load recording device, plus a "Demec" (Demountable) strain gauge with a gauge length of 6 ins (152.4 mm)

Test Cylinder
Laboratory Testing

(For making, curing and testing procedure see clause 5.2.2(c)
(i) 2 No 152.4 mm dia by 304.8 mm Long Cylinders Nos 7 and 8 cast on 17.10.78.
(ii) Modulus of Elasticity determined on 31.10.78 (ie 14 days after casting) and the results for the following "Loading" is shown in Appendix 11.1(a)

\[ \text{Min. load, specified as 1 MPa} = \frac{1 \times 152.4 \times \pi}{1000 \times 4} = 18 \text{ kN} \]

\[ \text{Max Load, specified as (C + 1) MPa} = \frac{(14.4 + 1) \times 152.4 \times \pi}{1000 \times 4} = 281 \text{ kN} \]

\[ C = \frac{43.1}{3} = 14.4 \text{ MPa} \]

(iii) Modulus of Elasticity determined on 14.11.78 (ie 28 days after casting) and the results for the "loading" is shown in Appendix 11.1(b).

\[ \text{Min Load specified as 1 MPa} = 18 \text{ kN} \]

\[ \text{Max Load, specified as (C + 1) MPa} = \frac{(16.3 + 1) \times 152.4 \times \pi}{1000 \times 4} = 316 \text{ kN} \]

\[ C = \frac{49.0}{3} = 16.3 \text{ MPa} \]

(e) Specific creep plus shrinkage strain of Test Cylinders

Instrumentation

(i) Creep rig which can accommodate two cylinders, and the load being applied through automatically controlled hydraulic system with provision for manual adjustment on the creep rig. The calibration of the force gauges had been carried out earlier by Mr Mackenzie of Department of Civil Engineering as part of other Tests to read 22 800 lb = 99 kN.

(ii) Creep test room where the temperature and humidity are automatically controlled at 23°C ± 1°C and 50% ± 4% Relative Humidity.
(iii) Demec (Demountable) strain gauge with a gauge length of 8 ins: (203,2 mm)

Laboratory Testing
(For making, curing and testing procedure see Clause 5.2.2(d)

General
(i) 4 No. 104,5 mm by 304,8 mm long cylinders cast on 17.10.78
(ii) Moulds stripped on 18.10.78
(iii) Wet cured at 100% Relative Humidity until 24.10.78
(iv) Then stored in creep test room at 23°C ± 1°C and 50% ± 4% Relative Humidity to date.

Creep (and Shrinkage) Specimens
(i) Creeps Specimens No. 5 and 6 placed in Creep rig. on 31.10.78, (ie 14 days after casting) and Demec gauge readings taken before any load applied.
(ii) Actual Dia of Specimen No. 5 is 104,7 mm and of Specimen No 6 is 104,2 mm, thus use a Mean dia of 104,45.
(iii) Creep specimens No 5 and 6 subjected at 12.30 on 31.10.78 to a load of 20 000 lb (86,84 kN) on the force gauge (this is equal to a mean pressure of $\frac{86,84 \times 10^3}{104,45^2} \times \frac{4}{\pi}$ $\approx 10,13$ MPa) and the Demec readings taken, to give the elastic shortening of Concrete on Load application.
(iv) Total strain (creep and shrinkage) reading then observed as recorded in Appendix 11.2(a).

Shrinkage Specimens
(i) First Demec gauge readings of Shrinkage specimens Nos 3 and 4 on 20.10.78 (ie 3 days after casting)
(ii) Shrinkage readings then observed as recorded in Appendix 11.2(b)
Strain Measurement
The strains were measured by means of a Demec (demountable) strain gauge (8 ins) and before each set of readings were taken, the strain gauge zero was checked against an 8 ins. Invarbar. Both the bar and the strain gauge were kept in the same room as the test specimens to ensure that no temperature correction was required.

(f) Shrinkage of Concrete to S.A.B.S.
Instrumentation
Rigid frame with a dial gauge, reading 100 divisions per revolution and each \( \frac{1}{2} \) division equal to 0,001 mm.

Laboratory Testing
(For making, curing and testing Procedure see Clause 5.2.2(e)
(i) 3 No 75 x 75 x 200 mm prisms cast on 17.10.78
(ii) Initial length of prisms determined on 20.10.78 as given below
(iii) Final length of prisms determined on 14.11.78 as given below

<table>
<thead>
<tr>
<th>DATE</th>
<th>STANDARD READING</th>
<th>PRISM NO</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>7</td>
</tr>
<tr>
<td>1st Reading</td>
<td>6,028</td>
<td>13,090</td>
</tr>
<tr>
<td>20.10.78</td>
<td>6,027</td>
<td>13,088</td>
</tr>
<tr>
<td>Average</td>
<td>6,028</td>
<td>13,089</td>
</tr>
<tr>
<td>1st Reading</td>
<td>6,034</td>
<td>12,122</td>
</tr>
<tr>
<td>14.11.78</td>
<td>6,035</td>
<td>12,122</td>
</tr>
<tr>
<td>Average</td>
<td>6,034</td>
<td>12,122</td>
</tr>
<tr>
<td>Change in Reading</td>
<td>0,006</td>
<td>0,167</td>
</tr>
<tr>
<td>Overall Length mm</td>
<td></td>
<td>199</td>
</tr>
</tbody>
</table>
5.2.4 Analysis of Laboratory Observations

(a) Compressive Strength of Test Cubes

(i) At 14 days after casting

From section 5.2.3(c), the compressive strength of the 3 cubes tested are 43.8; 41.8 and 43.7 MPa, giving a mean compressive strength at 14 days of 43.1 MPa.

(ii) At 28 days after casting

From section 5.2.3(c) the compressive strength of the 3 cubes tested are 49.6; 47.9 and 49.7 MPa, giving a mean compressive strength at 28 days of 49.1 MPa.

(b) Modulus of Elasticity (Static) of Test Cylinders

(i) At 14 days after casting

The results from section 5.2.3(d) are plotted in graphical form and shown in Appendix 11.1(c).

For cylinder No 7,  
\[ \text{slope of gauge reading } 7/I = 30.73 \times 10^3 \ \text{MPa} \]  \[ \text{slope of gauge reading } 7/II = 26.45 \times 10^3 \ \text{MPa} \]  \[ \text{Diff} = 4.27 \]  \[ \therefore \text{Average slope} = 28.59 \times 10^3 \ \text{MPa} \]  \[ \therefore 15\% \text{ of slope} = 4.29 \]

For cylinder No 8,  
\[ \text{slope of gauge readings } 8/I = 28.91 \times 10^3 \]  \[ \text{slope of gauge readings } 8/II = 27.73 \times 10^3 \]  \[ \text{Diff} = 1.18 \]  \[ \therefore \text{Average slope} = 28.32 \times 10^3 \ \text{MPa} \]  \[ \therefore 15\% \text{ of slope} = 4.25 \]

\[ \therefore \text{Average Modulus of Elasticity at 14 days} = 28.5 \times 10^3 \ \text{MPa} \]

(ii) At 28 days after Casting

The results from section 5.2.3(d) are plotted in graphical form and shown in Appendix 11.1(d).
For Cylinder No 7,
slope of gauge reading 7/I = 30.27 x 10^3 
\[ \text{slope of gauge reading 7/II = 32.82 x 10^3} \] $\text{Diff} \ 2.55$
\[ \therefore \text{Average slope} = 31.55 x 10^3 \text{ MPa} \therefore 15\% \text{ of slope} = 4.73 \]

For Cylinder No 8,
Slope of gauge reading 8/I = 32.64 x 10^3 
\[ \text{slope of gauge reading 8/II = 29.64 x 10^3} \] $\text{Diff} \ 3.00$
\[ \therefore \text{Average slope} = 31.14 x 10^3 \text{ MPa} \therefore 15\% \text{ of slope} = 4.67 \]
\[ \therefore \text{Average Modulus of Elasticity of 28 days} = 31.35 \times 10^3 \text{ MPa} \]

(c) \textbf{Specific creep plus shrinkage strain of Test Cylinders}

(i) \textbf{Total Strain of Concrete}
For the loaded specimens Nos 5 and 6 at each stage the change in total strain relative to the preceding stage has been calculated, and this value then averaged to give the "mean change in total strain between two adjacent readings" as calculated in Appendix 11.2(a).

(ii) \textbf{Shrinkage Strain}
For the unloaded specimens Nos 3 and 4 at each stage the change in shrinkage strain relative to the preceding stage has been calculated, and this value then averaged to give the "mean change in shrinkage strain between two adjacent readings" as calculated in Appendix 11.2(c) and graphically plotted to a logarithmic scale in Appendix 11.2(e).

From Appendix 11.2(e) it can be clearly seen that the curve of shrinkage strain vs age of concrete is practically of constant slope at $t = 830$ days and the curve can be projected with confidence to give a shrinkage strain at $t = 10000$ days of $60 \times 10^{-5}$
(iii) Creep Strain

The creep strain of the loaded specimens is the difference between the total strain observed on the loaded specimens less the shrinkage strain observed on the unloaded specimens, as calculated in Appendix 11.2(c). The total cumulative creep strain and specific creep from loading at age of 14 days after casting has also been calculated in Appendix 11.2(c) and the specific creep graphically plotted to a logarithmic scale in Appendix 11.2(d).

From Appendix 11.2(d) it can be clearly seen that the curve of specific creep vs. days under load is not yet flat at \( t = (830-14) \) but the curve can be projected with reasonable confidence to give a value of specific creep at \( t = 10000 \) of \( 105 \times 10^{-6} \)/MPa.

However if one accepts that the complete Creep-Time curve plotted to a logarithmic scale is roughly anti-symmetrical in shape as indicated in the CEB-FIP 1970 (12.44)* Recommendations, then the total ultimate creep is of the order of twice that at the point of contraflexure of the curve. From Appendix 11.2(d) point of contraflexure is at 75 days with a specific creep value of \( 62 \times 10^{-6} \) per MPa, thus giving a projected value at \( t=10000 \) days of \( 124 \times 10^{-6} \)/MPa. Using the actual graph (6) for coefficient \( k_t \) from the above Recommendations for a member thickness \( e_m = 5 \) cm equivalent to that of the specimens used, indicates that 95% of the ultimate creep has taken place in the first 1000 days. From Appendix 11.2(d), the specific creep value at \( t = 1000 \) is \( 99 \times 10^{-6} \)/MPa, giving a projected value of \( t =10000 \) days of \( \frac{99}{0.95} = 105 \times 10^{-6} \)/MPa.

From the above it can be concluded that the specific creep value at \( t = 10000 \) of \( 105 \times 10^{-6} \)/MPa obtained from the projected specific creep vs time curve is acceptable.
(iv) **Modulus of Elasticity**

For the loaded specimens No 5 and No 6 strain readings were taken on either specimen immediately before and straight after loading and the results recorded in Appendix 11.2(a).

In Appendix 11.2(c) the Modulus of Elasticity at the age of loading (14 days) has been calculated from these readings and found to be 28.2 MPa (which agrees with the value of 28.5 MPa at 14 days obtained in accordance with BS 1881: Part 5: 1970 (12.50(c)*) which is for specimens that are loaded and unloaded three times).

(d) **Shrinkage of Concrete to SABS**

The shrinkage strain is defined as:

\[
\text{Shrinkage Strain} = \frac{\text{change in length}}{\text{original length}} \times 100
\]

<table>
<thead>
<tr>
<th>PRISM NO.</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>8</td>
<td>9</td>
</tr>
<tr>
<td>Change in Length mm</td>
<td>167 x 0.001</td>
<td>152 x 0.001</td>
</tr>
<tr>
<td>Original Length mm</td>
<td>199</td>
<td>200</td>
</tr>
<tr>
<td>% Shrinkage Strain</td>
<td>0.08</td>
<td>0.08</td>
</tr>
<tr>
<td>Mean % Shrinkage Strain</td>
<td>0.087</td>
<td></td>
</tr>
</tbody>
</table>

5.2.5 **Summary and Comments**

(a) **Compressive Strength of Test Cubes**

The specified 28 day strength of the Mix is 40 MPa, and the average 28 day strength of the one set of three cubes was 49.1 MPa.

This correlates very well with 28 day strength of the cubes made on site, the strength of the site sets varied from 47.4 MPa to 53.9 MPa.
(b) **Modulus of Elasticity (static) of Test Cylinders**

(i) **At 14 days**

The Modulus of Elasticity was found to be 28.5 kN/mm². This value was confirmed later during the creep test loading where a value of 28.2 was obtained.

(ii) **At 28 days**

The Modulus of Elasticity was found to be 31.4 kN/mm²

(c) **Specific creep plus shrinkage strain of Test Cylinders**

(i) **Specific Creep**

The specific creep value at \( t = 10000 \) was found to be \( 105 \times 10^{-6} \) /MPa

This value is lower than had been expected for the Windhoek aggregates and is of the same order as obtained for good aggregates such as Reef Quartzites, all loaded at age of 14 days.

(ii) **Shrinkage Strain**

The shrinkage strain value at \( t = 3 \) to \( t = 10000 \) was found to be \( 60 \times 10^{-5} \).

This value is higher than had been expected for the Windhoek aggregates.

(d) **Coefficient of Thermal expansion**

The coefficient of thermal expansion of concrete was found to be \( 12 \times 10^{-6} /\circ\) (See Section 4.3.5(iv))

(e) **Shrinkage of Concrete to SABS**

The shrinkage strain was found to be 0.087%, which is far higher than expected and shows that based on small scale testing the Windhoek aggregates are unacceptable.

However structures constructed with concrete made with these aggregates have performed satisfactorily.

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**EXPERIMENTAL INVESTIGATIONS**
6. THEORETICAL INVESTIGATIONS

6.1 DESIGN CRITERIA

6.1.1 Introduction
At the design stage of a prestressed concrete bridge, an estimate of the elastic shortening, the creep and shrinkage of the concrete over the lifetime of the bridge must be made in order to be able to determine the required prestressing force.

The designer has recourse to various Codes which give guidance regarding the magnitude of these effects, but all codes require the designers to have a knowledge of certain basic factors such as age at loading, relative humidity, etc. While on the surface it may appear that the more factors a code takes into account, the more realistic are the predictions, this is not correct, since much of the information required in such Methods is not really known at the design stage.

6.1.2 Site Design Criteria

(a) General
As these calculations are being done retrospectively, two and a half years after the bridge was built, the actual data required for the prediction of creep and shrinkage are known.

(b) Concrete Mix (f)
Details of the actual concrete mix used (See Section 5.1.2(c)) are as follows:

(i) Specified 28 day strength of concrete = 40 MPa
(ii) Actual Mean 28 day cube strength of concrete in instrumented span = 50.5 MPa
(iii) Cement Content 445 kg
(iv) Water/cement ration = 0.45

(c) Elastic Modulus of Concrete (E)
As determined in Section 5.2.5(b) the Modulus of Elasticity (E) at 28 days was found to be 31.4 kN/mm²

(d) Relative Humidity (H)
(i) The Outside Relative Humidity
The monthly average relative humidities observed at the Windhoek City Met station are given in Appendix 11.4(a) and show that the average outside relative humidity for (a) the first 6 months after casting and (b) in the 2½ years since casting are 35% and 30% respectively.

(ii) The Relative Humidity Inside Box
These were only recorded for the first 3 weeks after stressing and are given in Appendix 11.3(e) and show that the average inside relative humidity was 27%.

(e) Temperature (T)
(i) Outside Temperature
The monthly average temperatures observed at the Windhoek City Met. Station are given in Appendix 11.4(b) and show that the average outside temperature for (a) the first 6 months after casting and (b) in the 2½ years since casting are 23°C and respectively 21°C.

(ii) The Temperature Inside Box
The average temperatures inside the box were only recorded for the first 3 weeks after stressing and are given in Appendix 11.3(e) and show that the average inside temperature was 26°C.
Theoretical Thickness \( (e_m) \)

(i) Cross-Section of Box as a Whole

The theoretical thickness \( e_m \) is defined as the quotient of area of the section divided by the semi-perimeter \( P/2 \) in contact with the atmosphere and based on the fact that in Section 6.2.4 it was found that the inside and outside average relative humidities were similar, the whole of the inside exposed perimeter will be included.

\[
e_m = \frac{\text{Area of cross section (cm}^2\text{)}}{\frac{1}{2} \text{perimeter in contact with Atmosphere (cm)}}
\]

\[
e_m = \frac{7,788 \times 10^4}{4 \times 16,78 \times 10^2} = 34 \text{ cm}
\]

(ii) Mid span Cross beam of structure

This has been calculated in Section 5.1.5(b) and found to be

\[
e_m (\text{beam}) = 27 \text{ cm}
\]

Secondary Reinforcement

(i) Cross-section of Box

The geometric percentage of secondary reinforcement

\[
p = \frac{\text{Area of secondary Steel (mm}^2\text{)}}{\text{Area of cross-section (mm}^2\text{)}}
\]

\[
p = \frac{39,626}{7,788 \times 10^6} \times 100 = 0.51\%
\]

(ii) Mid span cross-beam

\[
p = \frac{\text{area of secondary steel}}{\text{area of cross-section}} = \frac{3140 \times 10^2}{(1,190 \times 0,305) \times 10^6}
\]

\[
= 0.87\%
\]

(h) Age of Concrete

The prestressing force was applied 43 days after the concrete in the instrumented span was cast.
(i) **Concrete Stress at Neutral Axis**

<table>
<thead>
<tr>
<th>SECTION</th>
<th>0.1L</th>
<th>0.4L</th>
<th>0.75L</th>
<th>0.9L</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Prestressing Force = kN</td>
<td>38 800</td>
<td>40 600</td>
<td>40 200</td>
<td>39 200</td>
</tr>
<tr>
<td>Average Initial Prestressing Force = 39 800 and Average Initial Concrete Stress = 5.2 MPa</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Final Prestressing Force = kN</td>
<td>29 700</td>
<td>30 500</td>
<td>31 000</td>
<td>29 700</td>
</tr>
<tr>
<td>Average Final Prestressing Force = 30 500 and Average Final Concrete Stress = 3.9 MPa</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

6.2 THEORETICAL CALCULATIONS USING VARIOUS CODES

6.2.1 CF115 : 1959 (12.52)*

(a) **Modulus of Elasticity of Concrete**
Based on the 28 day cube strength of 51 MPa, from table 4, the Modulus of elasticity is 40 kN/mm²

(b) **Shrinkage of Concrete**
(i) For post-tensioning at between 2 to 3 weeks after concreting the subsequent shrinkage per unit length is $20 \times 10^{-5}$ (t14 to t10 000)

(ii) Half the total shrinkage takes place during first month after transfer and three-quarters of shrinkage takes place during first 6 months after transfer.

(iii) Since transfer actually took place 5 weeks after casting, based on the above the subsequent shrinkage would be $20 \times 10^{-5} - \frac{1}{4} \times \left(\frac{20 \times 10^{-5}}{2}\right) = 15 \times 10^{-5}$ (t43 to t10 000)

(c) **Creep of Concrete**
(i) Creep is proportional to the initial stress in the concrete

(ii) For post-tensioning at between two to three weeks after concreting, where the concrete strength at transfer is greater than 40 MPa, the specific creep is $36 \times 10^{-6}$ per MPa stress (t10 000)
(Based on average stress this increases to $36 \times 10^{-6} \times \left(\frac{5.2}{4(5.2 + 3.9)}\right) = 42 \times 10^{-6}$/MPa)

(iii) Half the creep take place in first month after transfer and three quarters of the whole creep takes place in the first six months after transfer.

6.2.2 C.E.B.-F.I.P : 1970 (12.44)*

(a) Modulus of Elasticity

Elastic (secant) Modulus = $0.9 \times 6.6 \sqrt{\text{fcu} \times 0.8}$ where $\text{fcu} =$ cube strength of concrete = 51 MPa at 28 days (The code actually refers to cylinder strength and the approximation of cylinder strength = $0.8 \times$ cube strength has been used).

$\therefore$ Elastic (secant) Modulus $E_{b28} = 0.9 \times 6.6 \sqrt{51 \times 0.8} = 38$ kN/mm$^2$

(b) Creep of Concrete

For a constant stress $\sigma_b$ creep strain is defined as

$$\varepsilon_f = \frac{\sigma_b}{E_{b28}} \times \frac{\varphi_t}{t}$$

$\therefore$ specific creep based on constant stress $\frac{\varphi_t}{E_{b28}}$

where $t = k_c k_d k_b k_e k_t$

(i) $k_c$ (environmental conditions)

From diag(2) for Rel Humidity of 30% $k_c = 3.3$

(ii) $k_d$ (age at loading)

From diag(3) for age at loading at 43 days and $T = 20^\circ C$ (actual average $21^\circ C$)

$k_d = 0.9$
(iii) $k_b$ (composition of concrete)

From diag (4) - R12.31 for $c = 445$ kg and $w/c = 0.45$

\[ K_b = 1.05 \]

(iv) $k_e$ (theoretical thickness)

From diag (5) - R12.31 for $e_m = 34$ cm

\[ k_e = 0.73 \]

(v) $k_t$ (variation as a function of time)

From diag (6) - R12.31 for $e_m = 34$ cm

\[ k_t = 1.0 \text{ for } t = 10,000 \text{ days} \]

(for $t = 10$ days; $k_t = 0.05$; for $t = 100$ days; $k_t = 0.25$; for $t = 10,000$ days; $k_t = 0.8$)

\[ \text{Specific creep strain} = \frac{\varepsilon_c}{E_b} = \frac{3.3 \times 0.9 \times 1.05 \times 0.73 \times 1}{38 \times 10^3} \]

\[ = 60 \times 10^{-6} / \text{MPa} \]

(Based on an initial stress that decreases, specific creep is $60 \times 10^{-6} \times (\frac{1}{5} \times (5.2 + 3.9)) = 53 \times 10^{-6} / \text{MPa}$)

(c) Shrinkage of Concrete

Shrinkage strain defined as

\[ \varepsilon_r = \varepsilon_c \cdot k_b \cdot k_e \cdot k_p \cdot k_t \]

(i) $\varepsilon_c$ (environment)

From diagram (2) - R12.32 for Rel Humidity of 30%

\[ \varepsilon_c = 47.5 \times 10^{-5} \]

(ii) $k_b$ (composition of the concrete)

as $k_b$ for creep = 1.05

(iii) $k_e$ (theoretical thickness)

From diag (4) - 12.32 for $e_m = 34$ cm

\[ k_e = 0.61 \]
(iv) \( k_t \) (variation as a function of time)

From diag (6) \(-R12.32\) for \( e_m = 34 \text{ cm} \) for \( t_{43} \) to \( t = 10000 \)

Shrinkage still to develop \( k_t = (1-0.14) = 0.86 \)

(for \( t = 10 \text{ days} \) \( k_t = 0.05 \); for \( t = 100 \text{ days} \) \( k_t = 0.28 \)

for \( t = 1000 \text{ days} \) \( k_t = 0.75 \))

(v) \( k_p \) (depends on reinforcing steel)

\[
\frac{k_p}{p} = \frac{100}{100 + np} \quad \text{where } p = 0.51\% \text{ and } n = 20
\]

\[
= \frac{100}{100 + 20 \times 0.51} = 0.91
\]

\[
\therefore \text{Shrinkage strain } \varepsilon_t = 47.5 \times 10^{-5} \times 1.05 \times 0.61 \
\times 0.86 \times 0.91
\]

\[
\therefore \varepsilon_t = 24 \times 10^{-5} \text{ for } t_{43} \text{ to } t = 10000
\]

6.2.3 CEB - FIP : 1978 (12.45)*

(a) General

In this code, the concrete strengths referred to are cylinder strength and the approximation of cylinder strength = 0.8 \times cube strength has been used to correct the cube strength used throughout this report to cylinder strength.

(b) Modulus of Elasticity

The longitudinal Modulus of deformation at 28 days, using \( f_{ck} = 50.5 \times 0.8 = 40 \text{ MPa} \). \( E_c 28 \) is = 35 kN/mm²

(c) Creep of Concrete

The creep coefficient \( \varepsilon(t, t_0) \) is defined from

\[
\varepsilon_c (t, t_0) = \frac{\sigma_0}{E_{c28}} \varepsilon(t, t_0)
\]

Where \( \varepsilon_c (t, t_0) \) denotes creep strain at time \( t \) under a constant stress \( \sigma_c \) applied at time \( t_0 \). The creep coefficient \( \varepsilon(t, t_0) \) is given by

\[
\varepsilon(t, t_0) = \beta_a (t_0) + \psi d \beta_d (t-t_0) + \psi_f \left[ \bar{\beta}_f(t) - \beta_f(t_0) \right]
\]
where \( \beta_d \) = 0,4

\[
\Phi_f = \Phi_{f1} \cdot \Phi_{f2}
\]

where \( \Phi_{f1} = 3,3 \) from table e1 col 3

where \( \Phi_{f2} = 1,45 \) from clause e1.6 and figure e2

\[
\Phi_f = 3,3 \times 1,45 = 4,79 \quad (h_0 = \frac{2Ac}{v} = 1 \times 340 = 340 \text{ mm})
\]

\( \beta_d (t-t_0) \) from figure e3 = 1,0

\( \beta_f (t) \) from figure e4 = 1,0

\( \beta_f (t_0) \) from figure e4 = 0,39

\[
\Phi(t, t_0) = 0,13 + 0,4 \times 1,0 + 4,79 (1,0 - 0,39)
\]

\( = 3,45 \)

\[
\text{specific creep} = \frac{\Phi(t, t_0)}{E c_{28}} = 3,45 \times 0,81 = 0,999 \times 10^{-3} = 99 \times 10^{-6} / \text{MPa}
\]

(using simplified equation \( \Phi(t, t_0) = 0,4 + \Phi_f (\beta_f (t) - \beta_f (t_0)) \)

\[
= 0,4 + 4,79 (1,0 - 0,39)
\]

\( = 3,32 \)

(d) \textbf{Shrinkage of Concrete}

The shrinkage strain in interval (t-to) is

\[ \varepsilon_s (t, t_0) = \varepsilon_{s0} (\beta_s (t) - \beta_s (t_0)) \]

where \( \varepsilon_{s0} = \varepsilon_{s1} \times \varepsilon_{s2} \) \( \varepsilon_{s1} = -0,00058 \) where from clause e 1.6 and fig e5 \( \varepsilon_{s2} = 0,81 \)

\[
\varepsilon_{s0} = - 58 \times 10^{-5} \times 0,81 = - 47,0 \times 10^{-5}
\]

\( \beta_s (t) \) from figure e6 = 1,0

\( \beta_s (t_0) \) from figure e6 = 0,18

\[
\varepsilon_s (t, t_0) = - 47,0 \times 10^{-5} \times (1-0,18) = 38 \times 10^{-5}
\]
6.2.4 BS 5400: 1978 (12.46)*

6.2.4.1 General Factors

(a) Comment

Only the "general" factors given in Clauses 7.8.2.3, 7.8.2.4 and 7.8.2.5 are considered.

(b) Modulus of Elasticity of Concrete

Based on the 28 day cube strength of 51 MPa, from table 2, the Modulus of Elasticity is 34 kN/mm².

(c) Shrinkage of Concrete

(i) For post-tensioning at between 1 to 2 weeks after concreting the subsequent shrinkage per unit length for Exposure (70% R.H.) is \(20 \times 10^{-5}\) for \(t_{14} + t = 10,000\).

(ii) Half the total shrinkage takes place during the first month after transfer and three-quarters of shrinkage takes place during first 6 months after transfer.

(iii) Using Appendix C to adjust for the fact that Relative Humidity was 30% and the age of loading was 43 days.

\[
\text{shrinkage strain} = 20 \times 10^{-5} \left(\frac{47.5}{27.5}\right) \left(\frac{1-0.15}{1-0.05}\right) = 31 \times 10^{-5} \quad t_{43} + t = 10,000
\]

(d) Creep of Concrete

(i) Creep is proportional to the initial stress in the concrete.

(ii) For post-tensioning at between 7 days and 14 days after concreting and for humid or dry conditions of exposure where the required cube strength at transfer is greater than 40 MPa, the specific creep is \(36 \times 10^{-6}\) per MPa.

(iii) Using Appendix C to adjust for the fact that Relative Humidity was 30% and the age of loading was 43 days.

\[
\text{specific creep to} \quad t = 10,000 = 36 \times 10^{-6} \times \left(2.2 \times 0.9\right) = 39 \times 10^{-6}/\text{MPa}
\]

(Based on average stress this increases to \(39 \times 10^{-6} \times \left(\frac{5.2}{4.8} (5.9 + 3.9)\right) = 44 \times 10^{-6}/\text{MPa}\).
6.2.4.2 **Accurate Factors**

(a) **Comment**

The more detailed information given in Appendixes B & C is used, which is essentially that given in the CEB-FIP: 1970 (12.44)* except that a more accurate allowance for the reduction in creep and shrinkage due to the presence of prestressed or slack reinforcement is made in accordance with the equations by Neville (12.63)*. Since for the purpose of this research the strains at the Neutral axis and not at the level of the prestressing steel are required, the equations given in Appendixes C4 and C5 could not be used and instead the equations given by Neville for axial shortening were used.

(b) **Modulus of Elasticity of concrete**

Based on the 28 day cube strength of 51 MPa, from Table 41 the Modulus of Elasticity is 34 kN/mm².

(c) **Creep of Concrete**

For a constant stress \( f_c \), final creep deformations

\[
\Delta_{cc} = \frac{f_c \cdot \phi}{E_{28}}
\]

\[\therefore \text{specific creep based on constant stress} = \frac{\phi}{E \cdot b_{28}}\]

where \( \phi = k_L, k_m, k_c, k_e, k_j \)

(i) **coefficient** \( k_L \) (environment)

from fig 8 for R.H. = 30% \( k_L = 3.3 \)

(ii) **coefficient** \( k_m \) (maturity)

from fig 9, for age of loading = 43 days \( k_m = 0.9 \)

(iii) **coefficient** \( k_c \) (composition)

from fig 10 for \( k_c = 445 \text{ kg and w/c = 0.45} \ k_c = 1.05 \)

(iv) **coefficient** \( k_e \) (theoretical thickness)

from fig 11 for \( h = 340 \text{ mm} \ k_e = 0.73 \)
(v) \( k_j \) (variation as a function of time)

from fig 12 for \( h_e = 340 \, \text{mm} \) for \( t = 10,000 \, \text{days} \)

\[ \phi = 3.3 \times 0.9 \times 1.05 \times 0.73 \times 1.0 = 2.28 \]

\[ \therefore \text{Specific creep} = \frac{\phi}{E_b} = \frac{2.28}{34 \times 10^3} = 67 \times 10^{-6} / \text{MPa} \]

(vi) Creep Correction for restraint due to reinforcement

(Neville (12.64)*

(1) Reduction coefficient due to (slack) secondary reinforcement (assumed symmetrically reinforced)

axial shortening \( y_\text{AA} = a_1 = \frac{1 + p_n \phi}{1 + \eta \phi} \)

reduction coefficient \( = 0.91 \)

where \( p = \frac{A_s}{A_c} = \frac{39626}{7.788 \times 10^6} = 0.0051 \)

\[ n_0 = \frac{E_s}{E_c} = \frac{200}{34} = 5.9 \]

\[ \phi = 2.28 \]

\[ \eta = 0.92 \]

(2) Reduction coefficient due to Prestressing Steel in One Face

From First Principles for a prestressed beam based on Neville

(A) Reinforced with Prestressing steel

\[ \epsilon_{\text{Reinf}} = \epsilon_1 + (\epsilon_2 - \epsilon_1) \frac{Y_1}{Y_1 + Y_2} \]

\[ = \epsilon_1 \frac{\phi}{1 + p_1 n_0 (1 + \phi)} + \left( \frac{\epsilon_2 - \epsilon_1}{1 + p_1 n_0 (1 + \phi)} \right) \frac{Y_1}{Y_1 + Y_2} \]

where \( Y_2 = \text{dist from N.A. to Unreinforced fibre 2} \)

\( Y_1 = \text{dist from N.A. to Prestressing steel 1} \)

\( \epsilon_2 = \text{strain in concrete at fibre 2} \)

\( \epsilon_1 = \text{strain in concrete at fibre 1} \)

\( \epsilon_r = \text{Axial shortening, singly reinforced} \)

where \( \epsilon_2 = \text{Initial concrete stress in fibre 2 (Outer Unreinforced)=2.74MPa (at 0.4L)=0.081}\times10^{-3} \)

\[ \epsilon_1 = \text{Initial concrete stress in fibre 1 (at steel level)} = \frac{7.9}{34 \times 10^3} \]

\[ \therefore \text{Single reinforced} \quad \epsilon_r = 0.319 \times 10^{-3} \]
(B) **Unreinforced**

\[
\bar{\varepsilon}_{\text{Cone}} = \frac{f_o}{E_o} \cdot \phi + f(T) \cdot \left(1 + \eta \phi\right)
\]

where \(f_o\) = initial stress in concrete at NA = 5.2 MPa (at 0.4L)

\(f_T = \) change in stress in concrete NA due to creep

\[\text{unreinforced } \bar{\varepsilon} = 0.349 \times 10^{-3} - 0.024 \times 10^{-3} = 0.325 \times 10^{-3}\]

\[\text{Axial shortening reduction coeff. = } \frac{\text{reinforced}}{\text{unreinforced}} \frac{\bar{\varepsilon}_r}{\bar{\varepsilon}_c} = 0.99\]

Specific creep for reinforced and prestressed concrete web

\[= \text{specific creep (unreinforced)} \times 0.91 \times 0.99\]

\[= 67 \times 10^{-6} \times 0.91 \times 0.99 \]

\[= 60 \times 10^{-6}/\text{MPa}\]

(d) **Shrinkage of Concrete**

Shrinkage strain defined as

\[\Delta c_s = k_L, k_c, k_e, k_j, k_z\]

(i) **Coefficient \(k_L\) (environment)**

For unreinforced concrete, from fig 13 for Rel Hum = 30% \(k_L = 47.5 \times 10^{-5}\)

(ii) **Coefficient \(k_c\) (composition of concrete)**

For \(c = 445\) kg and w/c = 0.45 from fig 10 \(k_c = 1.05\)

(iii) **Coefficient \(k_e\) (effective thickness)**

For \(he = 340\) mm from diag 14 \(k_e = 0.61\)

(iv) **Coefficient \(k_j\) (variation as a function of time)**

For \(he = 340\) mm, for \(t = 43\) to \(t = 10000\) from fig 12 \(k_j = (1-0.14) = 0.86\)

\[\Delta c_s\] excluding internal restraint coefficient \(k_2\) =

\[\text{free shrinkage } \Delta c_s = 47.5 \times 10^{-5} \times 1.05 \times 0.61 \times 0.86 = 26.2 \times 10^{-5}\]

(v) **Correction for restraint due to reinforcement (Neville 12.64)*

(1) **Reduction coefficient due to (slack) secondary reinforcement**

(assumed symmetrically reinforced)

axial shortening reduction coeff. = \(\gamma_{N.A.1} = (a_1) = \frac{1}{1+\eta_n(1+\eta \phi)}\)

\[\text{NA} = \frac{1}{1+0.0051x5.9(1+0.92x2.28)} \]

\[= 0.91\]

\[p = \frac{\text{As}}{\text{Ac}} = \frac{39626}{7788} = 6 = 0.0051\]

\[\eta_o = \frac{\text{Es}}{\text{Ec}} = \frac{200}{34} = 5.9\]

\[\phi = 2.28\]

\[n = 0.92\]

*Tyler (12.1) found this value to be = 0.75*

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**THEORETICAL INVESTIGATIONS**
(2) Reduction coefficient due to prestressing steel (singly reinforced)

Axial shortening reduction coefficient

$$\gamma_{NA} = \frac{1+p \rho \eta}{1+\eta} \frac{1+\eta}{y^2}$$

where \(A_{sp} = \text{Area of Prestressing Steel}\)

$$p = \frac{A_{sp}}{A_s} = \frac{22 \times 12 \times 132.7}{7,788 \times 10^{-6}} = 0,00047$$

$$\gamma_{NA}^2 = \frac{1+0.00047 \times 5.9 (1+0.92 \times 2.3) \times 0.49}{0.33}$$

$$= \frac{1+0.00047 \times 5.9 (1+0.92 \times 2.3) \times 0.49}{0.33} = 0.92$$

$$\phi = 2.3$$

$$\eta = 0.92$$

(NB. If \(Y1 = 0\) ie prestress at NA then \(\gamma_{NA}^2\) becomes 0.91)

Shrinkage strain for reinforced and Prestressed Centre Web

$$= \text{Free Shrinkage strain} \times 0.91 \times 0.92$$

$$= 26.2 \times 10^{-5} \times 0.91 \times 0.92 = 23 \times 10^{-5}$$

6.2.5 SABS 0100 : 1980 (12.68)*

(a) Modulus of Elasticity

Based on the 28 day cube strength of 51 MPa, Modulus of Elasticity is 34 kN/mm²

(b) Shrinkage of Concrete

(i) For Post-tensioning at between 1 to 2 weeks after concreting the subsequent shrinkage per unit length for Windhoek exposure (35% R.H) is 35 x 10⁻⁵ for t14 + t10 000

(ii) Using BS 5400 : 1978 Appendix C to adjust for the fact that the age of loading was 43 days.

Shrinkage Strain = 35 x 10⁻⁵ x \((1-0.15)_{\frac{1}{1-0.05}} = 31 \times 10^{-5}

\(t_{43} + t_{10} 000\)

(c) Creep of Concrete

(i) Creep is proportional to the initial stress in the concrete.

(ii) For post tensioning at between 7 days and 14 days after concreting and for humid or dry conditions for exposure where the required cube strength of transfer is greater than 40 MPa, the specific creep is 36 x 10⁻⁶/MPa.

(iii) Using BS 5400 : 1978 Appendix C to adjust for the fact that the age of loading was 43 days and adjusting to Average stress.

Specific creep \(t_{10} 000 = 36 \times 10^{-6} \times 0.9 \times (\frac{5.2}{1.2}) = 31 \times 10^{-6} / \text{MPa}\)
6.2.6 Laboratory Values Adjusted for Site Conditions

6.2.6.1 Using CEB:FIP: 1970 Factors

(a) Comment
The actual laboratory test results for concrete made with Windhoek aggregates as set out in Section 5.2 will be referred to, and then adjusted for actual Site Conditions using the CEB - FIP (1970) graphs.

(b) Modulus of Elasticity of Concrete
As determined in Section 5.2.5(b) the Modulus of Elasticity at 28 days was found to be 31.4 kN/mm²

(c) Creep of Concrete

(i) The basic specific creep of the test cylinders from Appendix 11.2(d) to \( \varepsilon \) 10 000 \( \varepsilon \) 10\(^{-6}\) MPa for constant stress

(ii) \( k_c \) (environmental conditions) using diag (2) - Clause R12.31(12.44)
- laboratory conditions: R.H. = 50% \( \therefore k_c = 2.85 \)
- site conditions: R.H. = 30% \( \therefore k_c = 3.3 \)
\( \therefore k_c \) adjustment to basic creep = \( \frac{3.3}{2.85} \)

(iii) \( k_d \) (age at loading) using diag (3) - R12.31 (12.44)*
- laboratory conditions: age loaded = 14 days, stored at 23°C \( \therefore k_d = 1.2 \)
- site conditions: age loaded = 43 days, stored at 21°C \( \therefore k_d = 0.9 \)
\( \therefore k_d \) adjustment to basic creep = \( \frac{0.9}{1.2} \)

(iv) \( k_b \) (composition of concrete) using diag (4) - R12.31 (12.44)*
- Laboratory and site conditions the same \( \therefore k_b \) adjustment = 1.0
(v) \( k_e \) (theoretical thickness) using diag (5) - R12.31 (12.44)*

Laboratory conditions \( e_m = \frac{\text{Area of cylinder}}{\frac{1}{4} \times \text{perimeter}} = \frac{\pi D}{4} \)

\[ \frac{D}{2} - \frac{10.45}{2} = 5.23 \text{ cm} \quad \therefore \quad k_e = 1.2 \]

Site conditions \( e_m \) (box) = 34 cm \quad \therefore \quad k_e = 0.73

\therefore \quad k_e \text{ adjustment} = \frac{0.73}{1.2} \]

(vi) \( k_t \) (variation as a function of time) using diag (6) - R12.32 (12.44)*

Laboratory and site conditions the same to \( t = 10,000 \) days

\[ \therefore \quad k_t \text{ adjustment} = 1.0 \]

Specific creep adjusted for site conditions for constant stress

Specific creep \[ 105 \times 10^{-6} \times \left( \frac{3.3}{2.85} \right) \times \left( \frac{0.9}{1.2} \right) \times \left( \frac{0.73}{1.2} \right) \]

\( (t_0 \rightarrow t_{10,000}) = 56 \times 10^{-6} / \text{MPa} \)

(d) Shrinkage of Concrete

(i) The basic shrinkage strain of the test cylinders

From Appendix 11.2(e) shrinkage strain from to to \( t_{10,000} = 60 \times 10^{-5} \)

(ii) \( \varepsilon_c \) (environment) using diag (2) - R12.32 (12.44)*

Laboratory conditions RH = 50% \quad \therefore \quad \varepsilon_c = 38 \times 10^{-5}

Site conditions RH = 30% \quad \therefore \quad \varepsilon_c = 47.5 \times 10^{-5}

\therefore \quad \varepsilon_c \text{ adjustment} = \frac{47.5}{38}

(iii) \( k_b \) (composition of concrete) using diag (4) - R12.31 (12.44)*

Laboratory and site conditions the same

\[ \therefore \quad k_b \text{ adjustment} = 1.0 \]

(iv) \( k_e \) (theoretical thickness) using diag (4) - R12.32 (12.44)*

Laboratory conditions \( e_m = 5.2 \text{ cm} \quad \therefore \quad k_e = 1.20 \)

Site conditions \( e_m = 34 \text{ cm} \quad \therefore \quad k_e = 0.61 \)

\[ k_e \text{ adjustment} = \frac{0.61}{1.20} \]

THEORETICAL INVESTIGATION
(v) 
\[ k_t \] (variation as a function of time) using diag (6) \( R12.31(12.44) \)
laboratory and site conditions the same to \( t = 10 \, 000 \)
\[ k_t \] adjustment = 1.0

(vi) 
\[ k_p \] (depends on reinforcing steel)
laboratory conditions, \( p = 0 \) \[ k_p = 1 \]
site conditions \( p = 0.51\% \) \[ k_p = \frac{100}{100 \times 10 \times p} = 0.91 \]
\[ k_p \] adjustment = \( \frac{0.91}{1.0} \)

(v) 
\[ k_t \] (variation as a function of time) using diag (6) \( R12.31(12.44) \)
laboratory conditions \( e_m = 5.2 \) \( t_{14} \) to \( t_{10 \, 000} \)
\[ k_t = (1-0.38) \]
site conditions \( e_m = 34 \) for \( t_{43} \) to \( t_{10 \, 000} \)
\[ k_t = (1-0.14) \]
shrinkage strain, adjusted for site conditions
\[ t_{43} \) to \( t_{10 \, 000} \)
\[ = 60 \times 10^{-5} \left( \frac{57.5}{38} \right) \left( \frac{0.61}{1.2} \right) \left( \frac{0.91}{1.0} \right) \left( \frac{0.86}{0.62} \right) = 48 \times 10^{-5} \]

6.2.6.2 Using BS5400 : Accurate factors

(a) Comment
The factors are the same as those used for the CEB:FIP 1970 Method, except that the effect of the reinforcing and prestressing steel is included.

(b) Creep of Concrete
All factors as for 6.2.5.1(c) plus reinforcement factors from section 6.2.4.2(c)
\[ \text{ie. Specific creep} = 56 \times 10^{-6}/\text{MPa} \times 0.91 \times 0.99 \]
to \( t = 10 \, 000 \) days
\[ = 51 \times 10^{-6}/\text{MPa} \]

(c) Shrinkage of Concrete
All factors as for 6.2.5.1(d) except that prestressing steel factors from section 6.2.4.2(d)
\[ \text{ie. shrinkage strain} = 48 \times 10^{-5} \times 0.9 \times 0.91 \]
\[ t_{43} \) to \( t_{10 \, 000} \)
\[ = 40 \times 10^{-5} \]
6.3 SUMMARY

6.3.1 Comparison of Theoretical values

Summary of site data on which creep and shrinkage coefficients are based.

Concrete Mix: Specified 28 day strength = 40 MPa
Actual 28 day strength = 50.5 MPa
Cement content (c) = 445 kg
Water/cement ratio k = 0.45
Relative humidity H = 30%
Theoretical thickness of Box e_m = 34 cm
Temperature + 21°C
Reinforcement in box Prestressing steel = 0.47%
Secondary steel = 0.51%
Age of loading (after casting) t = 43 days

<table>
<thead>
<tr>
<th>DESIGN CODE</th>
<th>MODULUS of ELASTICITY Eb 28 days</th>
<th>SHRINKAGE STRAIN FROM APPL OF PRESTRESSING t=43 to t=10 000 day</th>
<th>CREEP STRAIN TO t=10 000 day</th>
</tr>
</thead>
<tbody>
<tr>
<td>CP 115 : 1959</td>
<td>kN/mm²</td>
<td>15 x 10^{-5}</td>
<td>42 x 10^{-6}/MPa</td>
</tr>
<tr>
<td>CEB : FIP 1970</td>
<td>40</td>
<td>24 x 10^{-5}</td>
<td>60 x 10^{-6}/MPa</td>
</tr>
<tr>
<td>CEB : FIP 1978</td>
<td>38</td>
<td>38 x 10^{-5}</td>
<td>99 x 10^{-6}/MPa</td>
</tr>
<tr>
<td>BS 5400 1978</td>
<td>35</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(i) General factors</td>
<td>34</td>
<td>31 x 10^{-5}</td>
<td>44 x 10^{-6}/MPa</td>
</tr>
<tr>
<td>(ii) Accurate factors</td>
<td>34</td>
<td>23 x 10^{-5}</td>
<td>60 x 10^{-6}/MPa</td>
</tr>
<tr>
<td>SABS 0100 : 1980</td>
<td>34</td>
<td>31 x 10^{-5}</td>
<td>31 x 10^{-6}/MPa</td>
</tr>
<tr>
<td>Laboratory values</td>
<td>31,4</td>
<td>48 x 10^{-5}</td>
<td>56 x 10^{-6}/MPa</td>
</tr>
<tr>
<td>(i) Adjusted CEB-FIP:1970</td>
<td>31,4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(ii) Adjusted BS 5400: Accurate Method</td>
<td>31,4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* In fact defined as $\phi$ & not $\phi$ in this Code

6.3.2 Comment

The laboratory values adjusted for actual site conditions are as close a designer can get to the "probable" actual values of the bridge since they do include the effect of the aggregates which may have significant influence on the magnitude of creep and shrinkage strain as shown in Fig 4.15 of "Handbook on the Unified Code for structural Concrete" (12.65) and is not yet covered by any Code.
7. COMPARISON OF SITE, LABORATORY & THEORETICAL FACTORS

7.1 EVALUATION OF RESULTS OBTAINED

In order to be able to determine the creep and shrinkage factors by the various methods, each of which require the knowledge of certain basic factors, the actual site data as summarised in Section 6.3.1 has been referred to. However, it must be pointed out that not all the factors listed are necessarily required or allowed for by the individual codes.

7.1.2 Site Values of Actual Bridge

Evaluation of the values as listed in Section 5.6.1

(a) Elastic Strain of Concrete

Modulus of Elasticity E at 28 days assumed = 31.4 kN/mm² and based on unreinforced laboratory cylinders.

Tests by Tyler (12.1)* showed that there was no scale effect between the Modulus of Elasticity of unreinforced laboratory cylinders and actual bridge superstructures.

(b) Shrinkage Strain of Concrete

Projected shrinkage strain $t_3$ to $t_{10 000} = 38 \times 10^{-5}$; (giving for the deck concrete a shrinkage strain from $(1)t_{43}$ to $t_{10 000}$ of $33 \times 10^{-5}$ and $t_{14}$ to $t_{10 000}$ of $36 \times 10^{-5}$)

NB The above values are for the mid-span cross-beam reinforced with 0.87% steel, and the corresponding values of unreinforced concrete are as follows using the factors given in

(i) CEB - F.I.P : 1970

Shrinkage $t_{43}$ to $t_{10 000}$ of $= 33 \times 10^{-5}$ / $0.85 = 39 \times 10^{-5}$
(corresponding shrinkage $t_{14}$ to $t_{10 000}$ of $43 \times 10^{-5}$)
(ii) **BS 5400 : 1978 (Accurate Factors)**

Shrinkage $t_{43}$ to $t_{10\,000}$ of $33 \times 10^{-5}/0.87 = 38 \times 10^{-5}$
(corresponding shrinkage $t_{14}$ to $t_{10\,000}$ of $42 \times 10^{-5}$)

(c) **Creep Strain of Concrete**

Projected specific creep to $t_{10\,000} = 56 \times 10^{-6}/\text{MPa}$ for a
decaying prestressing force, giving a specific creep to
$t_{10\,000}$ of $64 \times 10^{-6}/\text{MPa}$ for the average prestressing force,
in both cases for concrete loaded at 43 days after casting.

**NB**

1. This is for the centre web of the deck, which is both prestressed
longitudinally and reinforced with secondary steel and the
corresponding values of unreinforced concrete for the average
prestressing force are as follows, using the factors given in

(i) **CEB - FIP 1970**

No adjustment in creep factors for percentage of steel present.
(corresponding specific creep for concrete loaded at 14 days
$= 64 \times 10^{-6} \times 1.2 = 85 \times 10^{-5}/\text{MPa}$

(ii) **BS 5400 : 1978 (Accurate factor)**

Specific creep to $t_{10\,000} = 64 \times 10^{-6} \times 1.2 = 95 \times 10^{-6}$
Loaded at 14 days $0.91 \times 0.99 = 95 \times 10^{-6}$

2. There is another correction that in theory is required to the specific
creep of concrete as observed for the centre web as follows:

(i) Total strain in fact measured for the centre web, which was
reduced by both the prestressing steel and 0.51% secondary
reinforcing steel from which was subtracted shrinkage strain
for mid-span centre web where there was 0.87% reinforcing steel.

(ii) Using CEB FIP, the correction factor for the centre web 0.51%
steel is 0.91 and for the mid-spn cross-beam 0.87% steel
is 0.85 and for all practical purposes can be assumed to
be the same as the difference is outside the accuracy of the
investigation.
7.1.3 Laboratory Values

1. Adjusted Using CEB-FIP 1970

The values are as listed in Section 6.3

(a) Elastic Strain of Concrete

The Modulus of Elasticity E at 28 days based on unreinforced test specimens.

(b) Shrinkage strain of Concrete

The laboratory test results on unreinforced specimens have been adjusted for site conditions as per graphs and a secondary reinforcing steel area of 0.51% has been allowed for.

(c) Creep Strain of Concrete

As for shrinkage except no allowance for effect of reinforcing.


(a) Elastic Strain of Concrete

The Modulus of Elasticity E at 28 days based on unreinforced test specimens.

(b) Shrinkage Strain of Concrete

The laboratory test results on unreinforced specimens have been adjusted for site conditions as per graphs and based on 0.47% prestressing steel and 0.51% secondary reinforcing steel in the web.

(c) Creep Strain of Concrete

As for shrinkage strain.

7.1.4 Theoretical Values

(i) CP 115 : 1959

Evaluation of the values as listed in Section 6.3

This code should only have been used in the United Kingdom as the designer cannot allow for different conditions of temperature, humidity etc. to those prevailing in the United Kingdom.
(ii) CEB-FIP: 1970

Evaluation of the values as listed in Section 6.3

(a) **Elastic Strain of Concrete**
Modulus of Elasticity $E$ at 28 days based on unreinforced test specimens.

(b) **Shrinkage Strain of Concrete**
This is based on a secondary reinforcing steel area of 0.51 in the centre web.

(c) **Creep Strain of Concrete**
Specific creep based on constant stress to $10,000$. No mention is made in code of the effect of reinforcing steel on creep and it can only be assumed that this reduction coefficient has been ignored, as this is on the safe side.

**NB** The effect of the aggregates is not covered by the Code.

(iii) CEB-FIP: 1978

Evaluation of the values as listed in Section 6.3

(a) **Elastic Strain of Concrete**
Modulus of Elasticity $E$ at 28 days on unreinforced test specimens.

(b) **Shrinkage Strain of Concrete**
No mention made in code of effect of reinforcing steel, but see note below.

(c) **Creep Strain of Concrete**
Specific creep, based on constant stress to $10,000$. No mention made in code of effect of reinforcing steel, but see note below.

**NB**

(1) No mention is made of the effects of either prestressing or secondary reinforcing steel in the sections of the Code where the creep and shrinkage strain formulas are given. However, in the section where losses of prestress due to shrinkage and creep at the tendon are given, the designer is given the choice of
two formulas, one which includes the effect of the area of prestressing steel and the other one does not.

The effect of the prestressing steel is included in the factor

\[ S = \alpha \frac{A_p}{A_i} \left(1 + \frac{A_i}{I_i} \cdot Y_{ip}^2 \right) \]

where \( \alpha = \frac{E_s}{E_c} = \frac{200 \times 10^3}{35 \times 10^3} = 5.7 \)

\[ \therefore S = 5.7 \times \frac{0.03495}{7.785} \left(1 + \frac{7.788 \times 0.71^2}{2586} \right) \]

\[ S = 0.06 \quad \text{(Area of tendons = 21x12x138.7 mm}^2 \]

\[ = 0.03495 \text{ m}^2 \]

\[ \therefore \text{This is approx. from Fig e7} \]

\[ \text{equal to } S = 0 \text{ which is the case of } A_p = 0 \]

\[ \therefore \text{Effect of prestressing steel is not significant} \]

\[ Y_{ip} = \text{Ecc. of Cable} = 0.71 \text{ at mid span.} \]

2. The effect of aggregates is not covered by the code.

(iv) BS 5400

General Factors

Evaluation of the values as listed in Section 6.3.

This code is still general in character, but allowance can be made for the important effect of humidity and the factors quoted certainly can be used as a preliminary guide.

Accurate Factors

Evaluation of the values listed in Section 6.3.

(a) Elastic Strain of Concrete

Modulus of Elasticity \( E \) at 28 days.

On unreinforced test specimens.

(b) Shrinkage Strain of Concrete

This is based on 0.47% prestressing steel and 0.51% reinforcing steel in the centre web.

(c) Creep Strain of Concrete

This is based on 0.47% prestressing steel and 0.51% reinforcing steel in the centre web.
This Code basically allows for most effects, except that the effect of Aggregate is not covered although the designer is warned of this effect and is referred to specialist literature on this subject.

(v) SABS 0100 : 1980
Evaluation of the values as listed in Section 6.3. This Code is general in character, but most important feature is that it recommends that specialist literature be consulted.

7.1.5 Actual Bridge Design Values

(a) General
When this bridge was being designed in 1973, the following creep and shrinkage factors were assumed, based on the results of tests conducted by Mackenzie (12.6)* on Port Elizabeth Aggregates, adjusted for Windhoek conditions using the CEB-FIP : 1970 graphs.

(b) Original Design Stage Data
Concrete Mix Specified 28 day Strength = 40 MPa
Cement Content (c) = 425 kg
Water/cement ratio(R) = 0.43
Relative humidity (H) = 30%
Temperature (T) = 20%
Reinforcement in box (P) = 0%
Age of loading after casting (t) = 14 days.

(c) Elastic Modulus of Concrete
The Elastic Modulus $E$ at 14 days was assumed to be $31 \times 10^3$ MPa and $E$ at 28 days to be $33.6 \times 10^3$ MPa

(d) Shrinkage Strain of Concrete
Shrinkage strain for $t_{14}$ to $t_{10\ 000} = 50 \times 10^{-5}$

(e) Creep Strain of Concrete
Creep factor $\theta$ to $t_{10\ 000}$ (loading at age of 14 days) = 3.0

$\therefore$ By CEB-FIP definition specific creep $= \frac{\theta}{E_{28}} = \frac{3}{33.6 \times 10^3} = 89 \times 10^{-6}$/MPa

COMPARISON OF SITE, LAB. & THEORETICAL VALUES
7.1.6 Effect of Reinforcing Steel

(a) General
At first glance, based on BS 5400:1978 where it states that "For structural members containing reinforcement, the magnitude of shrinkage and creep which can be realised is greatly reduced" it appears as if this factor should be taken into account, although if neglected, the answer would be on the safe side. The BS 5400:1978 recommendations regarding the effect of reinforcement are based on Neville (12.64)* who produces all the theoretical formulas, but gives no guidance as to when the effect of the reinforcement should be allowed for in calculations. As the CEB-PTP:1970 code only considers the effect of steel on shrinkage and the DIN 4227:1979 (12.60)* does not refer to the effect of the reinforcement, the problem was further investigated by referring to

(i) Leonhardt (12.66(b))*
In the lecture notes Volume 5, in section 17.43 it is stated that the reinforcement has a noticeable effect on creep and shrinkage if

\[ JS + Z > 0.05 \ J_T \]

where \( J_T \) = Moment of Intertia of Section

Substituting \( 0.028 > 0.05 \times 2.586 \)

which is incorrect

\[ 0.028 \leq 0.129 \]

and

\[ \therefore \text{Steel can be neglected.} \]

(ii) Leonhardt (12.61)*
In the book "Spannbeton für die Praxis" in Section 12.2 the effect of prestressing and reinforcing steel on the creep and shrinkage of concrete is covered in detail. It is stated that the effect of the steel should be included if the Moment of Inertia of all the steel relative to its common centre of gravity is appreciable.
(b) **Practical Design Approach**

It is general practice in bridge design when determining the estimated losses of prestress due to creep and shrinkage of concrete to ignore the effect of the prestressing and reinforcing steel as it is on the safe side, especially as the basic creep and shrinkage factors can only be "guestimated."

(c) **Experimental Investigation Approach**

In order (for the purpose of this investigation) to be able to compare the actual site values observed, which were measured at the Neutral Axis of the deck with the theoretical values, the effect of the prestressing and secondary reinforcing steels had to be allowed for.

The equations given in Appendices C4 & C5 of BS 5400 : 1978 (12.46)* are for the effect of the prestressing and reinforcing steel at the level of the prestressing steel and therefore could not be used directly. Instead reference had to be the equations given by Neville (12.64)* for the shortening at the Neutral axis which were then used in Sections 6.2.4.2 and 6.2.5.2.
### 7.2 Summary

#### 7.2.1 Tabulation of Values

<table>
<thead>
<tr>
<th>Factor</th>
<th>Original Assumed Design Data</th>
<th>Actual Site Data (used for Theoretical Values)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Mix: Specified 28 day Strength MPa</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>Actual 28 day Strength MPa</td>
<td></td>
<td>50,5</td>
</tr>
<tr>
<td>Cement Content (c) kg</td>
<td></td>
<td>425</td>
</tr>
<tr>
<td>Water/Cement Ratio (R)</td>
<td></td>
<td>0,43</td>
</tr>
<tr>
<td>Relative Humidity (H) %</td>
<td></td>
<td>20</td>
</tr>
<tr>
<td>Theoretical Thickness of box (e_m cm)</td>
<td>34</td>
<td>34</td>
</tr>
<tr>
<td>Temperature (t)°C</td>
<td></td>
<td>20</td>
</tr>
<tr>
<td>Reinforcement in box (Prestressing (Secondary Steel) %</td>
<td>0</td>
<td>0,47</td>
</tr>
<tr>
<td>Age of loading after casting t days</td>
<td>14</td>
<td>43</td>
</tr>
</tbody>
</table>

Table of the creep and shrinkage strains as obtained by the following methods:

<table>
<thead>
<tr>
<th>Method</th>
<th>Modulus of Elasticity at 28 days kN/mm²</th>
<th>Shrinkage Strain</th>
<th>Specific Creep Strain for Average Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Reinforced</td>
<td>Plain</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Concrete t=43</td>
<td>Concrete t=14</td>
</tr>
<tr>
<td></td>
<td></td>
<td>to t=10,000</td>
<td>to t=10,000</td>
</tr>
<tr>
<td>Actual Site Values</td>
<td>31.4</td>
<td>33×10⁻⁵</td>
<td>36×10⁻⁵</td>
</tr>
<tr>
<td>Design Codes</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CP 115 : 1959</td>
<td>40</td>
<td>15×10⁻⁵</td>
<td></td>
</tr>
<tr>
<td>CEB-FIP : 1970</td>
<td>38</td>
<td>24×10⁻⁵</td>
<td>25×10⁻⁵</td>
</tr>
<tr>
<td>CEB-FIP : 1978</td>
<td>35</td>
<td>38×10⁻⁵</td>
<td>44×10⁻⁵</td>
</tr>
<tr>
<td>MS 5400 : 1978:</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>(i) General Factors</td>
<td>34</td>
<td>31×10⁻⁵</td>
<td>32×10⁻⁵</td>
</tr>
<tr>
<td>(ii) Accurate Factors</td>
<td>34</td>
<td>23×10⁻⁵</td>
<td>24×10⁻⁵</td>
</tr>
<tr>
<td>SABS 0100 : 1980</td>
<td>34</td>
<td>31×10⁻⁵</td>
<td>35×10⁻⁵</td>
</tr>
<tr>
<td>Laboratory Values</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(i) Adjusted CEB-FIP:1970</td>
<td>31,4</td>
<td>48×10⁻⁵</td>
<td>50×10⁻⁵</td>
</tr>
<tr>
<td>(ii) Adjusted BS 5400:1978</td>
<td>31,4</td>
<td>40×10⁻⁵</td>
<td>42×10⁻⁵</td>
</tr>
<tr>
<td>Original Design Values</td>
<td>33,6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Based on Original Data)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Comparision of Site, Lab & Theoretical Values**
7.2.1 COMMENT

(a) Although throughout this research, reference has been made to the fact that the centre web was prestressed 43 days after the concrete was cast, other parts of the deck were in fact cast 16 days after the centre web and thus were only 24 days old at prestressing. In view of this we have also shown the adjusted creep and shrinkage strains for loading at 14 days after casting.

(b) The original design values as shown have not been adjusted for the average yearly Relative Humidity of 30% since the Relative Humidity varies considerably and it is advisable to choose a low value.

(c) The "original design" values compare well with the "adjusted actual site" values and also the adjusted "laboratory values" and certainly could not be reduced, as had been suggested by some authorities (See Section 5.5.1) and which led to this research project.

(d) In this instance the CEB-FIP : 1970 factors for creep strain are reasonable but the shrinkage strain factors are low, which can easily happen as the effect of the aggregate is not included in the equations. This confirms that if the Codes are used on their own for the determination of creep and shrinkage strains, this can lead to gross errors in design.
8.1 The aim of this investigation was to determine how the generally accepted design values of creep and shrinkage of concrete compared with actual creep and shrinkage of a concrete bridge deck.

8.2 In theoretical terms, creep and shrinkage of concrete are influenced by many factors, but in practical design only those factors that have significant effect can be considered.

8.3 Even for these basic factors, the data has to be "guestimated" at the design stage thus justifying the omission of those factors that have only a secondary effect on the creep and shrinkage of concrete.

8.4 From the literature survey carried out it is obvious that the subject of creep and shrinkage is a researcher's haven, but for the design of most structures only a realistic evaluation of creep and shrinkage is required.

8.5 This research has shown that the results of laboratory tests to ASTM C512-76 on specimens made with the actual aggregates and adjusted for actual site conditions give realistic predictions of the creep and shrinkage of actual concrete structures.

8.6 The CEB-FIP : 1970 or BS 5400 : 1978 (Accurate Method) Design Codes can be used to predict the creep and shrinkage effects of concrete if no testing of the actual concrete is carried out, but the aggregate effect is not included. These methods should only be used if the aggregate effect is known in broad terms.

8.7 As regards the Windhoek aggregates, the "original Design values" as given in Section 7.2.1, which have been used for the design of all the prestressed concrete bridges on the Windhoek Western Link Road compare well with the actual site values, confirming the design stage "guestimates".

8.8 The recently published SABS 0100:1980 represents an improvement on the BS 5400:1978 General Method only as far as shrinkage is concerned, but not as regards creep and is not as good as the two codes referred to in para 8.6 above.
9.1 TENTATIVE DESIGN RECOMMENDATIONS

This investigation has shown for the Windhoek aggregates that the design practice of predicting the elastic shortening, creep and shrinkage strains of concrete on the results of laboratory tests to ASTM C 512-76 on specimens made with actual aggregates and adjusted for actual site conditions using the CEB-FIP : 1970 graphs give realistic predictions of the creep and shrinkage of actual concrete structures.

9.2 In order to evaluate the aggregate effect, specimens made with the aggregates (and cements) used commonly throughout Southern Africa should be tested in accordance with ASTM C 512-76 and published for general use. In fact, considerable research has already been done in this regard, but to date only limited data has been made generally available (12.6)*.

9.3 Although it can be postulated with reasonable confidence that the conclusions reached in para. 7 also hold good throughout Southern Africa, this should be confirmed by investigation.
ACKNOWLEDGEMENTS

10.1 The author is especially grateful to Mr B J MacKenzie for his keen interest and encouragement and to the Laboratory staff of the Department of Civil Engineering, of the University of Cape Town for assistance with the laboratory work.

10.2 The author is also grateful to the following Hawkins & Osborn personnel:

Mr A Rust and the other members of the site staff for attending to the field measurements over the last three years.

The Secretaries for deciphering my handwriting and typing the thesis between their other work.

The draughting staff for tracing the diagrams and the printing department for the document production.

10.3 The author would like to thank the National Building Research Institute, Windhoek and the Weather bureau, Department of Transport for their co-operation.

10.4 The experimental investigation of the Brakwater South Bridge was carried out by permission of the then Director of Roads, South West Africa Administration.
## Appendix 11.1(b) Modulus of Elasticity: Concrete Cylinders at 28 Days

**Specimen No: 7**
- **Gauge Factor:** 1 Div = 1.02 x 10^-5 x $\frac{B}{6}$
- **Dia of Cylinder:** 6".
- **Date Tested:** 14/11/78
- **Pressure:** $\frac{N x 10^5}{x 10^{-5}} = KN x 0.054820$

### Table A

<table>
<thead>
<tr>
<th>Load (kn)</th>
<th>Stress (MPa)</th>
<th><strong>Gauge I</strong></th>
<th><strong>Gauge II</strong></th>
</tr>
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## Appendix 11.1(a) Modulus of Elasticity: Concrete Cylinders at 14 Days

**Specimen No: 7**
- **Gauge Factor:** 1 Div = 1.02 x 10^-5 x $\frac{B}{6}$
- **Dia of Cylinder:** 6".
- **Date Tested:** 31/10/78
- **Pressure:** $\frac{N x 10^5}{x 10^{-5}} = KN x 0.054820$

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## Appendixes

**Specimen No: 8**
- **Gauge Factor:** 1 Div = 1.02 x 10^-5 x $\frac{B}{6}$
- **Dia of Cylinder:** 6".
- **Date Tested:** 14/11/78
- **Pressure:** $\frac{N x 10^5}{x 10^{-5}} = KN x 0.054820$

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**Specimen No: 8**
- **Gauge Factor:** 1 Div = 1.02 x 10^-5 x $\frac{B}{6}$
- **Dia of Cylinder:** 6".
- **Date Tested:** 31/10/78
- **Pressure:** $\frac{N x 10^5}{x 10^{-5}} = KN x 0.054820$

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### APPENDIX 11.2

**CREEP AND SHRINKAGE - U.C.T. LABORATORY**

#### APPENDIX 11.2(a)

**ASTM TOTAL STRAIN MEASUREMENT : LOADED SPECIMENS - U.C.T. LABORATORY SHEET : T1**

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**ASTM TOTAL STRAIN MEASUREMENT : LOADED SPECIMENS - U.C.T. LABORATORY SHEET : T2**

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### APPENDIX 11.2(b)

#### ASTM SHRINKAGE CONTROL UNLOADED SPECIMENS - U.C.T. LABORATORY SHEET: S1

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#### APPENDIX 11.2(b)

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<th>STRAIN GAUGE NO</th>
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<th>SPECIMEN NO 4</th>
<th>A MEAN STRAIN GAUGE POINT 2</th>
<th>S Strain (x10^-5)</th>
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### APPENDIX 11.3

**CREEP AND SHRINKAGE - BRACKWATER SOUTH BRIDGE**

**TOTAL STRAIN MEASUREMENT: CENTRE WEB OF ONE SPAN (INCLUDES TEMP.)**

<table>
<thead>
<tr>
<th>Date</th>
<th>Time</th>
<th>Gauge Pt&quot;A&quot;-Underside top slab</th>
<th>Gauge Pt&quot;B&quot;-At Neutral Axis</th>
<th>Gauge Pt&quot;C&quot;-Top of Bottom Slab</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reading 1</td>
<td>Reading 2</td>
<td>Reading 3</td>
<td>Reading 1</td>
<td>Reading 2</td>
</tr>
<tr>
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<td>Reading 3</td>
<td>Reading 1</td>
<td>Reading 2</td>
</tr>
<tr>
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<td>Reading 2</td>
<td>Reading 3</td>
<td>Reading 1</td>
<td>Reading 2</td>
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**APPENDIX 11.3(a)**

**BRACKWATER SOUTH BRIDGE: WINDHOEK**

**TOTAL STRAIN MEASUREMENT: CENTRE WEB OF ONE SPAN (INCLUDES TEMP.)**

<table>
<thead>
<tr>
<th>Date</th>
<th>Time</th>
<th>Gauge Pt&quot;A&quot;-Underside top slab</th>
<th>Gauge Pt&quot;B&quot;-At Neutral Axis</th>
<th>Gauge Pt&quot;C&quot;-Top of Bottom Slab</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reading 1</td>
<td>Reading 2</td>
<td>Reading 3</td>
<td>Reading 1</td>
<td>Reading 2</td>
</tr>
<tr>
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<tr>
<td>Reading 1</td>
<td>Reading 2</td>
<td>Reading 3</td>
<td>Reading 1</td>
<td>Reading 2</td>
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## APPENDIX 11.3(a)

### BRAMATER SOUTH BRIDGE : WINDROCK

**TOTAL STRAIN MEASUREMENT : CENTRE WEB OF ONE SPAN (INCLUDES TEMP.)**

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<th>Date</th>
<th>Relative Time</th>
<th>Gauge Pt &quot;A&quot; Underside top slab</th>
<th>Gauge Pt &quot;B&quot; at Neutral Axis</th>
<th>Gauge Pt &quot;C&quot; Top of Bottom Slab</th>
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<tbody>
<tr>
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<td>796</td>
<td>747</td>
<td>792</td>
</tr>
<tr>
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<td>799</td>
<td>750</td>
<td>794</td>
</tr>
<tr>
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<td>18.11.78</td>
<td>778</td>
<td>725</td>
<td>765</td>
</tr>
</tbody>
</table>

* Before any longitudinal stressing was carried out

<table>
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<th>Date</th>
<th>Relative Time</th>
<th>Gauge Pt &quot;A&quot; Underside top slab</th>
<th>Gauge Pt &quot;B&quot; at Neutral Axis</th>
<th>Gauge Pt &quot;C&quot; Top of Bottom Slab</th>
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<tbody>
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<tr>
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<td>20.11.78</td>
<td>777</td>
<td>722</td>
<td>768</td>
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</table>

* After longitudinal stressing was completed

<table>
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<th>Relative Time</th>
<th>Gauge Pt &quot;A&quot; Underside top slab</th>
<th>Gauge Pt &quot;B&quot; at Neutral Axis</th>
<th>Gauge Pt &quot;C&quot; Top of Bottom Slab</th>
</tr>
</thead>
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<td>722</td>
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<td>21.11.78</td>
<td>20.11.78</td>
<td>776</td>
<td>722</td>
<td>768</td>
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<tr>
<td>22.11.78</td>
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<td>775</td>
<td>722</td>
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<td>722</td>
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### APPENDIX 11.3(b)

### BRAMATER SOUTH BRIDGE : WINDROCK

**TOTAL STRAIN MEASUREMENT : CENTRE WEB OF ONE SPAN (INCLUDES TEMP.)**

<table>
<thead>
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<th>Date</th>
<th>Relative Time</th>
<th>Gauge Pt &quot;A&quot; Underside top slab</th>
<th>Gauge Pt &quot;B&quot; at Neutral Axis</th>
<th>Gauge Pt &quot;C&quot; Top of Bottom Slab</th>
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</thead>
<tbody>
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<td>754</td>
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<td>13.2.79</td>
<td>774</td>
<td>717</td>
<td>755</td>
</tr>
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<td>13.2.79</td>
<td>773</td>
<td>715</td>
<td>754</td>
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<td>770</td>
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<td>775</td>
<td>714</td>
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<tr>
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<td>13.2.79</td>
<td>775</td>
<td>714</td>
<td>753</td>
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<th>Gauge Pt &quot;A&quot; Underside top slab</th>
<th>Gauge Pt &quot;B&quot; at Neutral Axis</th>
<th>Gauge Pt &quot;C&quot; Top of Bottom Slab</th>
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<td>740</td>
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### APPENDIX 11.3(a)

**BRAWATER SOUTH BRIDGE - WINDHOEK**

**TOTAL STRAIN MEASUREMENT : CENTRE WEB OF ONE SPAN (INCLUDES TEMP.)**

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<th>Date</th>
<th>Time</th>
<th>Age in days</th>
<th>Temp. Inside or Outside</th>
<th>Temp. Inside or Outside</th>
<th>Gauge Pt &quot;A&quot;</th>
<th>Gauge Pt &quot;B&quot; at Neutral Axis</th>
<th>Gauge Pt &quot;C&quot;</th>
<th>Gauge Pt &quot;C&quot; Top of Bottom Slab</th>
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<tr>
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### APPENDIX 11.3(a)

**BRAWATER SOUTH BRIDGE - WINDHOEK**

**TOTAL STRAIN MEASUREMENT : CENTRE WEB OF ONE SPAN (INCLUDES TEMP.)**

<table>
<thead>
<tr>
<th>Date</th>
<th>Time</th>
<th>Age in days</th>
<th>Temp. Inside or Outside</th>
<th>Temp. Inside or Outside</th>
<th>Gauge Pt &quot;A&quot;</th>
<th>Gauge Pt &quot;B&quot; at Neutral Axis</th>
<th>Gauge Pt &quot;C&quot;</th>
<th>Gauge Pt &quot;C&quot; Top of Bottom Slab</th>
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*Sheet No C5*
### APPENDIX 11.3(a)

**BRAWATER SOUTH BRIDGE: WINDHOEK**

**TOTAL STRAIN MEASUREMENT: CENTRE WEB OF ONE SPAN (INCLUDES TEMP.)**

<table>
<thead>
<tr>
<th>Date</th>
<th>Time</th>
<th>Age in Days</th>
<th>Temp. Inside</th>
<th>Temp. Outside</th>
<th>Relative Moisture</th>
<th>Relative Humidity</th>
<th>Gauge Pt &quot;A&quot; Underside Top Slab</th>
<th>Reading 1</th>
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#### APPENDIX 11.3(b)

**BRAWATER SOUTH BRIDGE: WINDHOEK**

**TOTAL STRAIN MEASUREMENT: CENTRE WEB OF ONE SPAN (INCLUDES TEMP.)**
### APPENDIX 11.3(b)

**BRAMWATER SOUTH BRIDGE : WINDHOEK**

**SHRINKAGE MEASUREMENT : MIDSPAN CROSS BEAM (INCLUDES TEMP.)**

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* Before transverse cables stressed

### APPENDIX 11.3(b)

**BRAMWATER SOUTH BRIDGE : WINDHOEK**

**SHRINKAGE MEASUREMENT : MIDSPAN CROSS BEAM (INCLUDES TEMP.)**

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* This reading was taken after all the longitudinal cables and one of the transverse cables had been stressed
### APPENDIX 11.3(b)

**BRAWMATER SOUTH BRIDGE : WINDHOEK**

**SHRINKAGE MEASUREMENT : MIDSPAN CROSS BEAM (INCLUDES TEMP.)**

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( ) Estimated

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### APPENDIX 11.3(b)

**BRAWMATER SOUTH BRIDGE : WINDHOEK**

**SHRINKAGE FOR EVALUATION OF ACTUAL CREEP STRAIN**

**MIDSPAN CROSS BEAM (INCLUDES TEMP.)**

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### Shrinking for Evaluation of Actual Creep Strain

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* Faulty
### APPENDIX 11.3(c)

**BRAKWATER SOUTH BRIDGE: WINDHOEK**

**Creep Strain Determination: Centre Web of one Span**

<table>
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<th>AGE UNDER LOAD</th>
<th>EXP. OUTSIDE BID</th>
<th>TEMP AT HEADING °C</th>
<th>SHrinkage &amp; TEMPERATURE CONTROL</th>
<th>NETT CREEP STRAIN AT NEUTRAL AXIS CAL.</th>
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<tr>
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<td>MAX SHINKAGE TEMPERATURE STRAIN (0.03x10^-5)</td>
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This reading was taken after any longitudinal 2,3.

+ This is Elastic Shortening plus some creep strain assumed.

### APPENDIX 11.3(c)

**BRAKWATER SOUTH BRIDGE: WINDHOEK**

**Creep Strain Determination: Centre Web of one Span**

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| 29.9.78  | 6.30 | 4 32 11 13 24 | 840 3 749 -5 833 +1 877 -3 948 -2 801 -4 | -2.7 | +1.7 | - 
| 30.9.78  | 6.35 | 5 34 8 9 27 | 838 -5 744 -10 830 -2 875 -5 948 -2 798 -7 | -5.2 | +6.0 | +0.8 
| 2.10.78  | 6.35 | 7 34 9 10 35 | 835 -8 741 -13 826 -6 871 -9 946 -4 796 -9 | -8.2 | -4 | +4.8 | -3.4 
| 3.10.78  | 6.35 | 8 32 8 9 35 | 838 -5 747 -7 830 -2 880 -0 947 -3 802 -3 | -3.3 | +5 | +6.0 | +2.7 
| 13.10.78 | 6.15 | 18 33 18 19 18 | 833 -10 746 -8 823 -9 874 -6 947 -7 800 -5 | -7.5 | +5 | -6.0 | -13.5 
| 27.10.78 | 6.15 | 32 36 12 13 34 | 829 -14 739 -15 818 -14 874 -6 937 -13 792 -13 | -12.5 | -1 | +1.1 | -11.3 
| 7.11.78  | 6.00 | 43 34 15 18 54 | 831 -12 742 -12 819 -13 874 -6 941 -9 800 -5 | -9.5 | +5 | -6.0 | -15.5 
| 11.11.78 | 6.55 | 47 30 15 18 80 | 830 -13 743 -11 817 -15 871 -9 935 -15 793 -12 | -12.5 | +5 | -6.0 | -16.5 
| 17.11.78 | 6.05 | 53 36 18 21 29 | 835 -9 746 -8 830 -11 880 -0 942 -8 803 -2 | -6.2 | +7 | -6.4 | -14.6 
| 27.11.78 | 6.05 | 63 37 17 23 50 | 830 -13 743 -11 874 -15 874 -6 934 -16 802 -3 | -10.7 | +9 | +10.0 | +21.5 
| 11.12.78 | 6.05 | 77 36 14 23 27 | 832 -11 743 -11 876 -14 876 -4 936 -14 803 -2 | -9.7 | +8 | -5.6 | -19.2 
| 20.12.78 | 6.00 | 87 37 17 22 64 | 831 -12 743 -11 876 -15 876 -4 937 -13 804 -1 | -8.2 | +10 | -12.0 | -20.1 
| 3.1.79   | 6.00 | 100 39 20 24 12 | 835 -8 744 -10 877 -14 877 -3 937 -13 804 -1 | -8.2 | +10 | -12.0 | -19.1 
| 13.1.79  | 6.00 | 110 37 16 23 57 | 835 -8 743 -11 877 -14 877 -3 938 -12 803 -2 | -6.3 | +9 | -10.9 | -19.1 
| 23.1.79  | 6.00 | 120 33 20 21 57 | 835 -8 743 -11 875 -15 876 -5 936 -14 803 -2 | -9.5 | +7 | -8.4 | -17.9 
| 2.2.79   | 6.00 | 130 33 18 20 73 | 835 -8 742 -12 874 -16 874 -6 936 -14 801 -4 | -10.0 | +6 | -7.2 | -17.2 
| 13.2.79  | 6.00 | 141 36 17 19 90 | 834 -9 742 -12 875 -16 875 -5 937 -13 802 -3 | -9.7 | +5 | -6.0 | -15.7 

APPENDIX 3(d)

BRAEKWATER SOUTH BRIDGE: WINDHOEK

ACTUAL SHRINKAGE STRAIN DETERMINATION: MIDSPAN CROSS BEAM

<table>
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<tr>
<th>Date</th>
<th>Time</th>
<th>Age in Days</th>
<th>Temperature Outside 0°C</th>
<th>Actual Strain Strain (including Temperature Strain) x 0.094 x 10⁻⁵</th>
<th>Axial Temp. Strain</th>
<th>Nett Strain</th>
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</thead>
</table>
| 22.9.79  | 6.00 | 150 26 17 19 90 | 834 -9 741 -13 815 -17 573 -7 935 -15 802 -3 | -10.7 | +6 | +6.0 | +16.7 
| 23.3.79  | 6.00 | 177 32 16 19 63 | 832 -12 737 -17 813 -19 871 -9 932 -18 798 -7 | -13.7 | +6 | -6.0 | -19.7 
| 27.4.79  | 6.00 | 214 30 9 9 65 | 832 -11 737 -17 813 -19 872 -8 932 -18 799 -6 | -13.2 | +5 | +6.0 | -7.2 
| 29.6.79  | 7.00 | 277 22 6 7 35 | 814 -29 719 -35 796 -34 855 -25 917 -33 774 -31 | -31.2 | -7 | -8.4 | +22.8 
| 18.9.79  | 8.00 | 358 32 8 9 (9) | 822 -21 729 -25 805 -27 937 -13 778 -27 | -22.6 | -5 | +6.0 | -16.6 
| 19.10.79 | 7.35 | 369 34 6 7 (7) | 827 -16 733 -21 807 -25 930 -20 777 -26 | -22.0 | -7 | -8.4 | +13.6 
| 6.11.79  | 6.30 | 409 39 15 20 (20) | 823 -20 730 -24 804 -28 927 -23 775 -30 | -25.0 | +6 | -7.2 | +32.2 
| 12.12.79 | 5.25 | 443 38 20 24 (24) | 826 -15 737 -17 806 -26 932 -18 777 -28 | -20.8 | +10 | -12.0 | +32.8 
| 20.3.80  | 7.30 | 552 35 15 18 (18) | 818 -25 730 -24 795 -37 925 -25 772 -33 | -28.8 | +4 | +4.8 | +33.6 
| 19.6.80  | 7.40 | 643 23 6 7 (7) | 805 -38 714 -40 790 -42 925 -25 766 -39 | -36.8 | -7 | -8.4 | +28.4 
| 8.1.81   | 7.14 | 846 36 19 23 (23) | 821 -22 735 -19 805 -27 932 -18 787 -16 | -20.8 | +9 | +10.8 | +31.6 
| 24.3.81  | 7.12 | 921 26 11 14 (14) | 813 -30 725 -29 797 -35 924 -26 783 -22 | -28.4 | 0 | 0 | +28.4 |
APPENDIX 11-3(e) RELATIVE HUMIDITY AND TEMPERATURE OF AIR INSIDE BOX

HUMIDITY MEASUREMENTS BY NBR1
APPENDIX 11.3 G1 SHINKAGE STRAIN OF CONCRETE OF BRKAWATER SOUTH BRIDGE VS TIME AFTER CASTING
APPENDIX H-3: CREEP STRAIN AT N.A. OF CONCRETE OF BRIDGEM AS TIME UNDER LOAD.

SECTION AT 0.9 L

SECTION AT 0.75 L

SECTION AT 0.4 L

SECTION AT 0.1 L

CREEP STRAIN (£10^-3)

TIME UNDER LOAD IN DAYS

0 1 2 3 4 5 6 7 8 9 10 20 30 40 50 60 70 80 90 100 200 300 400 500 1000 2000 3000 4000 5000 10,000 DAYS

18.5 x 10^-5 (CEB)

35 x 10^-5 (CEB 1970)

24 x 10^-5 (CEB 1970)

30 x 10^-5 (CEB 1970)

24 x 10^-5 (CEB 1970)

20 x 10^-5 (CEB 1970)

This part of the curve is abnormal and will be ignored.
### APPENDIX 11.5  
**BRACKWATER SOUTH BRIDGE - SPECIAL TEMP EFFECT READINGS**  
Page 11.5-1  
Sheet No 12

**APPENDIX 11.5**

**BRACKWATER SOUTH BRIDGE : WINDHOEK**  
**TOTAL STRAIN MEASUREMENT : CENTRE WEB OF ONE SPAN (INCLUDES TEMP.)**

<table>
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<th>Time</th>
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<th>LOCATION OF SECTION AT 0.3L</th>
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### APPENDIX 11.5

**SPECIAL READINGS THROUGHOUT ONE DAY TO DETERMINE TEMP. EFFECTS**  
**BRACKWATER SOUTH BRIDGE : WINDHOEK**  
**SHRINKAGE MEASUREMENT : MIDSPAN CROSS BEAM - (INCLUDES TEMP.)**

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<td></td>
<td>16.00</td>
<td>29</td>
<td>30</td>
<td>29</td>
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<td>839</td>
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<td>18.00</td>
<td>29</td>
<td>26</td>
<td>42</td>
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<td>838</td>
<td>-1</td>
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</tbody>
</table>
DETERMINATION OF THE COEFFICIENT OF THERMAL EXPANSION OF CONCRETE CYLINDERS

1. Shrinkage Control Specimens No. 1 and No. 2 (which are similar to shrinkage specimens Nos. 4 and 5 described in Section 5.2) and have been stored in creep test room at 24°C and 50% Rel Humidity since 24-10-78.

2. All strains were read use a Demec Strain gauge as used for other tests described in section 5.2.

3. Temperature of air in oven was obtained by means of a thermometer placed with cylinders in oven.

<table>
<thead>
<tr>
<th>DATE</th>
<th>STRAIN GAUGE ZERO READING (x10^-5)</th>
<th>TEMPERATURE OF AIR IN ROOM/OVEN</th>
<th>SPECIMEN 1</th>
<th>SPECIMEN 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>GAUGE POINT 1 (x10^-5)</td>
<td>GAUGE POINT 2 (x10^-5)</td>
<td>GAUGE POINT 3 (x10^-5)</td>
<td>GAUGE POINT 1 (x10^-5)</td>
</tr>
<tr>
<td>Specimens read in creep, room at 24°C as recorded on Thermometer</td>
<td></td>
<td></td>
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<tr>
<td>21-5-81</td>
<td>800,0</td>
<td>Room 24°C</td>
<td>820,0</td>
<td>803,0</td>
</tr>
<tr>
<td>Specimens placed in oven on 21-5-81 and expansion read on</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25/5/81</td>
<td>800,0</td>
<td>Oven 34°C</td>
<td>829,5</td>
<td>815,5</td>
</tr>
<tr>
<td>Specimen kept in oven until 26-5-81 and then returned to creep room</td>
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<tr>
<td>26/5/81</td>
<td>800,0</td>
<td>Room 34°C</td>
<td>830,5</td>
<td>815,5</td>
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<tr>
<td>Specimens kept in creep room at 24°C</td>
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</tr>
<tr>
<td>9/6/81</td>
<td>800,0</td>
<td>Room 24°C</td>
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<td>803,0</td>
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<tr>
<td>Mean of creep room readings at 24°C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>800,0</td>
<td>Room 24°C</td>
<td>820,0</td>
<td>803,0</td>
<td>813,0</td>
</tr>
<tr>
<td>Mean of Oven readings at 34°C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>800,0</td>
<td>Oven 34°C</td>
<td>830,0</td>
<td>815,5</td>
<td>825,8</td>
</tr>
<tr>
<td>∴ Dif</td>
<td>+ 10,0</td>
<td>+ 12,5</td>
<td>12,3</td>
<td>16,3</td>
</tr>
<tr>
<td>∴ Mean/strain/10° rise</td>
<td>11.6 x 10^-6/10°C</td>
<td>14.1 x 10^-5/10°C</td>
<td></td>
<td></td>
</tr>
<tr>
<td>∴ Average strain/°C = (11.6 + 14.1) x 10^-5/°C = 12.9 x 10^-6/°C of cylinders</td>
<td></td>
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</tbody>
</table>
When it is necessary to determine the deformation of the concrete due to shrinkage at some stage before the ultimate condition is reached, it may be assumed that half the total shrinkage takes place during the first month after transfer and the three-quarters of the total shrinkage takes place in the first six months after transfer.

d. Loss of prestress due to creep of the concrete. The loss of stress in the prestressing steel due to creep of the concrete should be calculated on the assumption that creep is proportional to the initial stress in the concrete for the permissible stressing recommended in this Code. The loss of stress is obtained from the product of the module of elasticity of the steel as given in Clause 304 e and the creep of the concrete adjacent to the prestressing steel. Usually it is sufficient to assume in calculating this loss that the tendons are located at their centroid.

For pre-tensioning where the specified minimum cube strength for the concrete at transfer is greater than 40 N/mm², the creep of the concrete per unit length should be taken as 48 \(\times 10^{-6}\) per N/mm². For lower values of specified minimum cube strength at transfer the creep per unit length should be assumed to be 48 \(\times 10^{-6}\) \(\times \frac{40}{u_t}\) per N/mm².

For post-tensioning at between two and three weeks after concreting where the specified minimum cube strength at transfer is greater than 40 N/mm², the creep of the concrete per unit length should be taken as 36 \(\times 10^{-6}\) N/mm². For lower values of specified minimum cube strength at transfer the creep per unit length should be taken as 36 \(\times 10^{-6}\) \(\times \frac{40}{u_t}\) per N/mm².

The figures given in the preceding paragraphs relate to the limiting creep after several years. When it is necessary to determine the deformation of the concrete due to creep at some earlier stage, it may be assumed that half the total creep takes place in the first month after transfer and that three-quarters of the total creep takes place in the first six months after transfer.

f. Loss of prestress during anchoring. In post-tensioning systems allowance should be made for movement of the steel at the anchorage when the prestressing force is transferred from the tensioning equipment to the anchorage. The loss due to this movement is particularly important in short members, and for such members the allowance made by the designer for this loss should be checked on the site.

g. Losses of prestress due to steam curing. Where steam curing is employed in the manufacture of prestressed concrete units by the 'long-line' method of pre-tensioning, it may be necessary to allow for a further loss of prestress. See Clause 501 g.
R 12.21 Determining the elastic modulus (See Supplement C 12.22)

R 12.221 DEFLECTIONS UNDER INSTANTANEOUS OR RAPIDLY CHANGING LOADS
In the presence of instantaneous or rapidly changing loads, the elastic modulus tangential to the origin, $E_{m}$, of the concrete at an age of $j$ days may be evaluated using the following formulae.

(a) Normal aggregate concretes:

$$E_{m} = 66,000 \sqrt{K_{12} K_{13}}$$

where $K_{12}$ is the average compressive strength of the concrete at $j$ days, measured on cylinders.

(b) Lightweight aggregate concretes:

$$E_{m} = 18,000 \sqrt{K_{12} K_{13} \gamma}$$

where $\gamma$ is the volumetric weight of the concrete.

For structural lightweight aggregate concretes, the values of $E_{m}$ are subject to a large scatter; the above formula is thus only a guide. It is advisable to deduce $E_{m}$ from tests on cylindrical specimens (see R 12.11).

R 12.222 DEFLECTIONS UNDER PROLONGED LOADS (cf. R 44.12 and R 44.13)
For normal aggregate concretes and lightweight aggregate concretes, use may be made of the secant modulus in areas with working stresses equal to the tangent modulus defined above, less 10%.

For the determination of internal forces produced by creep (R 12.31), the elastic modulus is the secant modulus $E_{se}$, defined for an age $j = 28$ days.

R 12.3 DEFERRED DEFORMATIONS OF THE CONCRETE (SHRINKAGE AND CREEP)
The coefficients given in R 12.31 (creep) and R 12.32 (shrinkage) form a working basis and are valid only for Portland cement concretes of normal quality, hardening under normal conditions and subject to working stresses at the most equal to 40% of their rupture stress. (See Supplement C 12.3.)

This refers to the average rupture stress on cylinders at the particular time.

R 12.31 Creep

(1) In order to evaluate the order of magnitude of deflected deformations due to creep under working conditions, use may be made of the theory of linear creep. For a constant stress $\sigma_c$, this theory leads to a calculation of the final creep deformation of the formula:

$$\varepsilon = \frac{\sigma_c}{E_{se}} \phi$$

In this formula, $E_{se}$ is the value of the secant modulus of the concrete at an age of 28 days (R 12.22) which gives an indication of the quality of the concrete and $\phi$ is a coefficient covering the particular working conditions envisaged. This coefficient is equal to the product of two partial coefficients

$$\phi = \phi_0 \phi_1 \phi_2 \phi_3 \phi_4$$

where:

$\phi_0$ depends on the environmental conditions,

$\phi_1$ depends on the hardened concrete at the age of loading,

$\phi_2$ depends on the composition of the concrete,

$\phi_3$ depends on the theoretical thickness of the member,

$\phi_4$ covers the development of the deflected deformation with time.

The value of $\phi_4$ calculated with the values given below of these different coefficients is an average value. When creep has a large influence on the limit state under consideration, an increase or a reduction of the order of 15% should be considered, so as to cover the most unfavourable case.

If the stresses producing creep are themselves influenced by creep or if they vary in a continuous manner, it is necessary to use iterative methods or to revert to appropriate analytical methods.

Where creep has a very large effect on the stresses, it may be advantageous to produce curves giving $\phi_0$ and $\phi_1$ from equations.

(2) Coefficient $k_0$ (environmental conditions).

(3) Coefficient $k_1$ (hardening at the age of loading).
The degree of hardening of the concrete at the age of loading exerts an influence at least as big as the climatic conditions.

The values in the diagram above refer to Portland cement, hardened under normal conditions, i.e., at an average concrete temperature of 20°C and with protection against excessive losses of moisture.

If the concrete hardens at a temperature other than 20°C, the age at loading is replaced by the corresponding degree of hardening:

\[ D = \sum (T + 10) \]

where \( D \) represents the degree of hardening at the moment of loading.

\[ D \text{ represents the number of days during which hardening has taken place at } 70°C. \]

(4) Coefficient \( A \) (composition of the concrete).

![Diagram](image1)

(5) Coefficient \( B \) (theoretical thickness).

![Diagram](image2)

The theoretical thickness \( e_{th} \) is the quotient of the area of the section \( B \) divided by the semi-perimeter \( p/2 \) in contact with the atmosphere. If one of the dimensions of the section

under consideration is very large with respect to the others, the theoretical thickness corresponds almost to the actual thickness.

If the dimensions are not constant along the member, an average theoretical thickness can be defined by paying particular attention to those sections in which the stresses are maximum.

(6) Coefficient \( k_s \) (variation as a function of time).

The values in the diagram refer to normal aggregate except for lightweight concretes. The final creep deformation should be deduced from tests carried out in accordance with the methods laid down by RILEM. Alternatively, it may be calculated by giving attention to normal aggregate concretes and in multiplying the result obtained by 1.6, or

\[ e_{th} = \frac{1}{2} \frac{d + (d + s)}{k_s} \]

where \( d \) depends on the environment; \( s \) depends on the composition of the concrete; \( k_s \) depends on the theoretical thickness of the member (see R 12.3.2); and \( k_s \) depends on the geometric percentage of longitudinal reinforcement of area \( A \) with respect to the cross-sectional area of the member \( B \):

\[ k_s = \frac{100}{A + p} \]

where \( A = 20 \) with regard to the effects of creep;

\( k_s \) defines the development of shrinkage as a function of time.

As a general rule, \( e_{th} \) as a function of \( k_s \) gives the reduction in length of the fibre at the centre of gravity of the tendon \( p \) under consideration.

The average values of the coefficients \( e_{th} \), \( k_s \), \( k_s \), and \( k_s \) as functions of the parameters which define them, may be taken from the following diagrams which are valid only for concretes which have been protected from excessive losses of moisture in their early days.

(2) Coefficient \( k_t \) (environment).

For unreinforced concrete, the average values of \( e_{th} \) can be taken from the following diagram:

(6) It has been shown experimentally that shrinkage of structural lightweight concretes lies between one and two times that of normal aggregate concretes with the same compressive strength.

R 12.4 POISSON'S RATIO

For elastic strains under normal working stresses, Poisson's ratio is taken as 0.2. For certain calculations, the effect of transverse expansion may be neglected.

R 12.5 COEFFICIENT OF THERMAL EXPANSION

Between 0°C and 100°C, the coefficients of thermal expansion of reinforced concrete containing a normal percentage of reinforcement and of prestressed concrete are taken as equal to:

- \( 1 \times 10^{-5} \) for normal aggregate concretes.
- \( 0.8 \times 10^{-5} \) for lightweight aggregate concretes.

Tests have shown that these coefficients may vary fairly widely. The measured coefficients vary from:

- \( 0.7 \times 10^{-5} \) to \( 1.3 \times 10^{-5} \) for normal aggregate concretes.
- \( 0.5 \times 10^{-5} \) to \( 1.1 \times 10^{-5} \) for structural lightweight aggregate concretes.

For these two types of concrete, the parameters which have any effect on the value of the coefficient of thermal expansion are: the nature of the cement, the nature of the aggregates, the mix, the hygroscopy, and the dimensions of the sections.

As far as normal aggregates are concerned, the lower values are obtained with calcareous aggregates and the highest values with siliceous aggregates.
The distinction between 'creep' and 'shrinkage' is conventional: normally, the delayed strains of loaded or unloaded concrete should be considered as different aspects of a single physical phenomenon.

For the relaxation of concrete, which is not considered in this Appendix, refer to the CEB-FIP Manual 'Structural effects of time dependent deformations in concrete'.

1.2 FIELD OF APPLICATION

This Appendix deals with the creep and shrinkage of concrete subjected to a compressive stress not exceeding 0.4 fct (at age t) and allowed to harden in constant ambient conditions.

It is accepted that its scope also extends to concrete in tension.

2.3 CREEP

2.3.1 Assumptions

Within the range of service stresses, the creep deformations due to stress fractions applied at different or equal intervals of time are considered to be additive (assumption of superposition).

Taking this into account, the creep deformation under constant stress is linear in relation to the stress; if we refer conventionally to the initial deformation under load imposed at 28 days, the creep coefficient $\varepsilon_0$ at $t_0$ can be defined from the equation:

\[ \varepsilon_0(t) = \frac{\varepsilon_0(t_0)}{E_{lt}} \cdot \sigma(t) \]  

where $\varepsilon_0(t_0)$ denotes the creep strain at time $t_0$ under a constant stress $\sigma$, $E_{lt}$ is the longitudinal modulus of deformation at 28 days (2.5.2 of the Code, Table 3.3).

The total strain at the instant $t$ under constant stress (Initial strain at the instant $t_0$ plus creep deformation), is given by:

\[ \varepsilon_{tot}(t) = \varepsilon_{0}(t_0) + \epsilon(t_0) + \frac{1}{E_{lt}} \epsilon(t) \]  

where $E_{lt}(t_0)$ denotes the initial value of the longitudinal modulus of deformation at age $t_0$.

\[ \]
4.1.5 CORRECTED AGE

To take account of the ambient temperature during the hardening of the concrete if it is appreciably different from 20°C and of the type of cement, the actual age of the concrete must be corrected.

For each actual period \( \Delta t \) during which the ambient mean temperature is \( T_{\text{av}} \), the corrected age is obtained from the following formula:

\[
\text{t} = \frac{a}{30} \left( T_{\text{av}} - 10 \Delta t \right)
\]

where:
- \( a \) is a coefficient which assumes the following values:
  1. for normal and slowly hardening cements,
  2. for rapid-hardening cements,
  3. for rapid-hardening high-strength cements;
- \( T \) is the mean daily temperature of the concrete in degrees C;
- \( \Delta t \) is the number of days when the mean daily temperature has assumed the value of 10°C.

4.1.6 NOTIONAL THICKNESS

The notional thickness is defined by:

\[
h_{\text{nt}} = \frac{1}{2} \Delta A
\]

where:
- \( \Delta A \) is a coefficient depending on the ambient environment (Table 4.1, column 3);
- \( A \) is the area of the concrete section;
- \( \Delta \) is the perimeter in contact with the atmosphere.

4.2 STRUCTURAL EFFECTS

4.2.1 GENERAL

In general, the effects of creep should be taken into account only for the serviceability limit states.

However, creep can sometimes occur and have an influence on behaviour at the ultimate limit state. This applies to second-order effects where creep improves the initial eccentricity, and for structures where the time dependent deformations are large in relation to the deformability of some of their elements. (For example, the effect of differential compression deformations of the columns in a tall building on the stress conditions in the floors they support.) It also applies to the larger prestress oscillations where lower is creep can increase the amplitude of the oscillations of the stresses.

---

For a more complete treatment of the problems, refer to the CEB/FIP Manual "Structural effects of the time dependent deformations in concrete".

The serviceability limit states referred to are:
- the limit state of deformation when the delayed strains in the concrete increase the deflections,
- the limit state of cracking when the effect of the delayed strains in the concrete is to vary the stress condition.

The delayed deformations in the concrete should be considered as imposed deformations modifying the initial configuration of strains and/or stresses in structures.

This influence is usually important only in the service condition because of the small deformability of the materials in the elastic phase. On the other hand, as for any other imposed strain, it is considerably reduced when an ultimate limit state is approached, as a result of the large inelastic strains of the concrete and of the steel.
The use of methods of numerical integration is generally necessary.

The application of formula (e.11) or (e.12) is limited to cases where the history of the stress is known and quite simple. In general, this is only rarely so and it is then necessary to use instead of (e.11) and (e.12), algebraic expressions obtained by introducing approximations (e.2.3.1) or by adopting a greatly simplified rule for the creep function (e.2.3.2).

2.2 GENERAL FORMULA

In applying the superposition principle (see e.1.3.1) the total deformation of the concrete is:

1. for a non-uniform stress variation

\[ \varepsilon (t, t_0) = \varepsilon(t, t_0) + \varepsilon (t, t_0) + \frac{d \varepsilon_o(t, t_0)}{dt} \frac{d \varepsilon_o(t, t_0)}{dt} \]  

2. for a uniform stress variation

\[ \varepsilon (t, t_0) = \varepsilon(t, t_0) + \frac{d \varepsilon_o(t, t_0)}{dt} \]  

where:
- \( \varepsilon(t, t_0) \) is the strain independent of stress (shrinkage, thermal expansion, etc.),
- \( \varepsilon_o(t, t_0) \) is the creep function (e.13),
- \( \frac{d \varepsilon_o(t, t_0)}{dt} \) is the variation of stress at the instant \( t_0 \),
- \( \varepsilon(t, t_0) \) is the strain variation in the infinitely small time interval \( dt \).

2.3 SIMPLIFIED FORMULAE

2.3.1 Use of algebraic equations

2.3.1.1 Mean stress method with variable \( E_0 \)

When the variation of stress is limited and does not exceed 30% in the time interval \( t_0 \) (e.g. evaluation of the losses of prestress) we can take:

\[ \varepsilon(t, t_0) = \varepsilon(t, t_0) + \frac{d \varepsilon_o(t, t_0)}{dt} \left( \frac{1}{2} + \frac{1}{E(t_0) \varepsilon_o(t_0)} \right) \]  

where \( \varepsilon_o(t_0) \) can be determined once and for all depending on the stress history and not subject to the low of the following type:

\[ \varepsilon(t, t_0) = \varepsilon(t, t_0) + \varepsilon_o(t, t_0) \]  

Here \( \varepsilon_0 \) and \( \varepsilon_1 \) are constants and can be determined once and for all depending on the stress history (see table 3 in the Manual). It can be accepted also as an adequate approximation that the \( \varepsilon_0 \) will not depend on the stress history even if \( \varepsilon_1 \) is often the same as the stress variations only follows the expression (e.13) approximately.

For conditions near the stress variations and the longitudes of modulation of formation given opposite, substituting the expression for \( X \) would lead to the result (e.16), the "mean stress".

2.3.1.2 Mean stress method with constant \( E_0 \)

If the variation of stress after the number of the longitudinal modulus of deformation can be neglected, formula (e.12) can be simplified to:

\[ \varepsilon(t, t_0) = \varepsilon(t, t_0) + \frac{d \varepsilon_o(t, t_0)}{dt} \frac{1}{2E(t_0) \varepsilon_o(t_0)} \]  

2.3.2 Total modulus method

When the stress does not vary or vary only very little (e.g. calculation of the stress under constant imposed stress, losses of prestress in sections with a low percentage of steel), we can take:

\[ \varepsilon(t, t_0) = \varepsilon(t, t_0) + \frac{d \varepsilon_o(t, t_0)}{dt} \frac{1}{2E(t_0) \varepsilon_o(t_0)} \]  

2.4 ASSESSMENT OF LOSSES OF PRESTRESS

When the prestressing tendons are sufficiently close to each other for them to be treated as a single tendon, then the losses of prestress can be assumed sufficient closely with the following formulae.

2.4.1 Losses due to shrinkage and creep

The formula (e.22) has been obtained by using the method of the mean stress method with constant \( E_0 \) (e.12) and by taking as equal the variations in the extensions of the steel and of the concrete at the level of the centre of gravity of the tendons.

The numerator term represents the sum of the effects of creep and shrinkage without taking into account the variations in extension in the same sequences of phenomena.

The reduction due to these interferences is represented by the denominator term which is positive and larger than 1.

The stress \( \sigma_{P_0} \) occurs if prestressing is carried out after the concrete had hardened (post-tensioning). In the case of pre-tensioning the stress to be taken into account is that which acts immediately after the release of the anchorage.

\[ \sigma_{P_0} = \frac{E(t, t_0) \varepsilon_o(t, t_0) + \frac{d \varepsilon_o(t, t_0)}{dt}}{1 + \frac{d \varepsilon_o(t, t_0)}{dt}} \]  

where \( \sigma = \sigma(x, y) \).

\[ \sigma_{P_0} \] denotes initial stress in the tendon due to the prestressing alone.

\[ \sigma_{P_0} \] denotes initial stress in the concrete at the level of the tendon due to self-weight and other permanent actions.
Where the variations \( \Delta \sigma_{\text{eq}} \) of the permanent loads intervene at successive time intervals \( t_i \), a term should be added to the numerator of the formula, viz
\[
\Delta \sigma_{\text{eq}} \psi(1, t_i)
\]

The formula [e.23] has been obtained by imposing observance of internal compatibility (plane section) and by adopting the simplified law for creep (eq. [e.18]); the tendons are treated as an internal elastic tie. An analogous procedure is generally applied (see the relevant literature) for the case of an external elastic tie (imposition of external compatibility) (e.g., a tied arch). In that case the formula [e.24] should be slightly modified and the ratio \( \delta \) should be calculated in an appropriate manner (see the Manual).

\[ e.2.4.1.2 \text{ Formula obtained from the simplified creep law} \]
\[
\Delta \sigma_{\text{eq},t} = \sigma_0 (t_i - 1) \sigma_{\text{eq}} -(t_p - 1) \frac{1 - \delta}{\delta} \sigma_{\text{eq},0} + \varepsilon_i (1-\delta) \varepsilon_i (t_i, t_p) E_i
\]
where \( t_p \) and \( t_i \) are three coefficients given by figures e.7, e.8 and e.9 depending on \( \psi(t, t_p) \) and the ratio \( \delta \) of the stiffnesses:
\[
\delta = \frac{A_p}{A_i} \left( 1 + \frac{A_i}{A_p} \frac{1}{y_i} \right)^2
\]
where:
- \( A_p \) denotes the sectional area of the tendons
- \( A_i, l_i \) denotes the area and the moment of inertia of the converted section
- \( y_i \) denotes the distance of the tendon from the centroid of the converted section.

\[ e.2.4.2 \text{ Taking account of the relaxation of the steel} \]
An assessment of the total loss of prestress due to shrinkage, creep and relaxation of the steel, taking account of the interdependence of these three phenomena, can be carried out by means of the following formula:
\[
\Delta \sigma_{\text{pr},t} = \frac{\varepsilon_i (t_i, t_p) E_i + \Delta \sigma_{\text{pr}} + \sigma_{\text{eq}, t}(t_i, t_p) \sigma_{\text{eq}} + \sigma_{\text{eq},0}}{1 - \alpha \frac{\sigma_{\text{eq},0}}{\sigma_{\text{eq}}}[1 + \psi(t, t_p) / 2]}
\]
where:
- \( \Delta \sigma_{\text{pr},t} \) denotes the variation of the tension in the tendons (negative) due to relaxation acting alone; the value to be taken into account is that which would occur under an initial tension of:
\[
\sigma_{\text{pr}} = \sigma_{\text{pr},0} - 0.3 \Delta \sigma_{\text{pr},t}
\]
where:
- \( \sigma_{\text{pr},0} \) initial stress in the tendon due to prestress and to permanent actions;
- \( \Delta \sigma_{\text{pr},t} \) denotes an a priori value of the total loss which should be checked subsequently with the results of formula [e.25] (iterative process).
APPENDIX 11.10
EXTRACTS FROM BS 5400 : 1978 (12.46)*

| Table 33: Shrinkage of concrete
| --- |
| Age of concrete (days) | Shrinkage (µm/m)
| Pre-curing: - moisture at 100% RH for 14 days | 100% - 140μm/m | 300% - 160μm/m |
| Post-curing: - moisture at 100% RH for 14 days | 100% - 140μm/m | 300% - 160μm/m |

BS 5400: Part 4:1978
The degree of hardening of the concrete at the age of loading may vary as an influence on stress or the environment.

The age of the concrete at the time of loading, $A$, and the age at which the test is performed can influence the strength of the concrete at the age of loading. For example, at 28 days, the test is performed on the concrete without additional influences, and the test is performed on the concrete with additional influences at the age of loading.

In general, the test is performed on the concrete without additional influences at the age of loading. For example, at 28 days, the test is performed on the concrete without additional influences, and the test is performed on the concrete with additional influences at the age of loading.

The degree of hardening of the concrete at the age of loading may vary as an influence on stress or the environment.

The degree of hardening of the concrete at the age of loading may vary as an influence on stress or the environment.
8. Creep and shrinkage

8.1 General notes on creep and shrinkage

By creep is meant the increase of permanent deformation with time caused by the application of constant stresses. Creep can be observed both in concrete and in steel. However, as distinguished from creep in steel, which is first revealed under stresses exceeding a definite limit, and which practically ends after the lapse of only a few days if held sufficiently distant from the tensile strength, concrete will creep under any load and for several years.

Shrinkage of concrete is understood as the changes of length which occur as the concrete dries out. The deformation of concrete caused by the setting heat is not covered by the term shrinkage and must be specially considered if its influence is considerable.

Creeping of steel and concrete and shrinkage of concrete alter the originally applied pre-stressing which, as a result, is generally reduced. This can eventually be outweighed by re-tensioning.

8.2 Creep in steel

Creep in steel need not be allowed for if either the steel is stressed within a range below the creep limit or the creep is eliminated by taking suitable measures, e.g. by re-tensioning the steel.

8.3 Creep in concrete

Creep in concrete depends on the hardened condition of the concrete at the moment of load application, also on the intensity of the stresses applied and the duration of their action, on the condition of the concrete, and on the moisture condition prevailing in its surroundings (see Table 5). On the assumption that the deformation of the concrete, while creeping, is proportional to the active stress, and on the assumption of a constant modulus of elasticity $E_b$, the fundamental relation between deformation and constant stress is:

$$\varepsilon = \frac{\sigma}{E_b} (1 + \phi) = \frac{1}{E_b} \varepsilon + \phi$$  \hspace{1cm} (1)

where

$$\phi = \frac{\text{creep}}{\text{elastic extension}} = E_b \cdot \varepsilon$$ \hspace{1cm} (2)

signifies the ratio of the specific change of length caused by creep to the elastic portion of the specific change in length (dimensionless creep value).

Furthermore, $\phi_b$ signifies the specific change of length caused by unit stress as a result of creep (unit of creep)$^2$, dimension (cm$^2$/kg). Both $\phi$ and $\phi_b$ are dependent on time and tend to attain their ultimate values $\phi_{\infty}$ and $\phi_{\infty b}$ respectively.

Table 5 contains the ultimate creep values for unreinforced concrete after creeping has finished ($t = \infty$, i.e. for practical purposes after about 4 years). If increment values are interpolated, upper and lower limit values shall likewise be chosen for these. In the evaluation of tests, also, fluctuations of the same order shall be taken into account.

In the case of heavy structural members whose smallest dimension is not less than 0.75 m, the creep value may be reduced by 10%, with a smallest dimension of not less than 1.5 m, by 20%. The effect of a specially strong or very one-sided unstressed bar reinforcement shall be allowed for in the calculation.

### Table 5: Ultimate creep value of unreinforced concrete for $t = \infty$

<table>
<thead>
<tr>
<th>Line</th>
<th>Location</th>
<th>Ultimate creep value $\phi_{\infty}$</th>
<th>Ultimate shrinkage value $\phi_{\infty b}$ (min. value)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>In water</td>
<td>0.050 to 1.000</td>
<td>0.0</td>
</tr>
<tr>
<td>2</td>
<td>In very humid air, e.g. directly over water</td>
<td>1.50 to 2.000</td>
<td>10</td>
</tr>
<tr>
<td>3</td>
<td>Generally in the air</td>
<td>2.00 to 3.000</td>
<td>20</td>
</tr>
<tr>
<td>4</td>
<td>In dry air, e.g. in dry inner rooms</td>
<td>2.50 to 4.000</td>
<td>30</td>
</tr>
</tbody>
</table>

The coefficients $k$ in Table 5 take account of the influence of the hardening condition of the concrete at the moment the stresses causing the creep are produced; they can be derived from Fig. 2 for the ratio $W / W_{\infty}$ which depicts the hardening condition. $W$ is the cube strength when the stress occurs, $W_{\infty}$ is the ultimate strength.

The ultimate strength $W_{\infty}$ of the concrete may be assumed as 1.3 $W_{28}$ with the use of cement Z 275, as 1.15 $W_{28}$ with cement qualities Z 375 and Z 475. For $W_{28}$, the value found in conducting the service test shall be substituted.
If the stress causing creep is altered at a later date, the effect of the additional stress shall be separately calculated. That k-value must be used for this which corresponds to the time of the stress alteration. The effects of creep resulting from the stress first applied and the later additional stress shall then be superimposed. If the load or the additional stress is applied at a late stage of concrete ageing, the following can be assumed with the use of Portland cement under normal hardening conditions for the coefficient k:

- after a hardening period of 3 months: \( k = 0.75 \)
- after a hardening period of 6 months: \( k = 0.65 \)

### 8.4 Shrinkage of the concrete

With tensioning before the concrete hardens (see Section 1.51), the influence of shrinkage in unreinforced concrete shall be taken into account with the basic values \( e_s \) given in Table 5. With tensioning after the concrete hardens (see Section 1.51), the minimum values \( e_s \) according to Table 5 shall be multiplied by 0.6 of the value k in Fig. 2, Section 8.3, penultimate paragraph, applies analogously to the determination of k.

It may be assumed in the calculation that the phenomenon of shrinkage in respect to time is proportional to creep.

The values given apply to structural members with a minimum dimension of 20 cm. For thinner structural members, they must be increased by 25%. For thick structural members with a minimum dimension of 75 cm, they may be reduced by the same amount.

For the calculation of intersecting forces of lattice girders which outwardly are statically indeterminate, the shrinkage value of Table 5 shall be substituted by means of k if the effect of shrinkage is not decreased by special measures (e.g. by contraction joints).

### 8.5 Proof of the effect of creep and shrinkage in concrete

The reduction of the tensile force under the influence of creep and shrinkage in concrete shall as a rule be mathematically proved.

The mathematical proof shall be carried out for all permanently acting stresses, i.e. primarily for the effect of pre-stressing and of the steady load. If an appreciable part of the live load acts permanently, the average amount of live load present shall also be treated as steady load.

The influence of creep and shrinkage shall likewise always be taken into account in the calculation of the intersecting forces of lattice girders which outwardly are statically indeterminate.

The proof shall be carried out for the condition stated in Section 9.34 with the greater limit value of the creep value according to Table 5, provided the smaller value does not produce in special cases more unfavourable stresses.

The fluctuations of the creep value between the limit values given shall likewise be taken into account in evaluating the deformation of the lattice girders which is to be expected.

If the continually acting loads are applied at an appreciably later time than the pre-stressing, such as can happen, for example, with precast structural members, the reduction of tension in the pre-stressing elements can in certain circumstances be substantially increased in consequence of the counter-effect coming into play later. This shall be allowed for by means of appropriate assumptions (see Section 8.3).

A proof based on tests is permissible in place of the mathematical proof if the test conditions for all influences mentioned in Section 8.1 are sufficiently consistent with the conditions on the building site, the upper and lower limits of the amount of creep are taken into account, and a State Materials Testing Institute conducts the tests.
Bild 1. Auswertung der Kriech- und Schwindergebnisse an der Luitpoldbrücke Landshut

Bild 2. Auswertung der Kriech- und Schwindergebnisse an der Güntherbrücke Ulm

Bild 3. Auswertung der Kriech- und Schwindergebnisse an der Lechauer Straßenbrücke Augsburg
Bild 4. Auswertung der Kriech- und Schwindergebnisse an der Werderbrücke Pforzheim

Bild 5. Auswertung der Kriech- und Schwindergebnisse an der Rheinbrücke Wiesbaden
Bild 8. Auswertung der Kriech- und Schwindergehene an der Brücke Mühlsbach München

Bild 9. Auswertung der Kriech- und Schwindergehene an der Eisenbahnhbrücke Harren
Bild 11. Auswertung der Kriech- und Schwindergebiete an der Berliner Bank, Berlin

Figure 10: Movements at centre of span, Mancunian Way.

Figure 11: Movements at centre of span, Western Avenue.
Figure 23: Mean shrinkage in precast and in situ slabs and companion specimens for Western Avenue.
Figure 17: Movements at centre of span, Sutton Lane Bridge.

Figure 18: River Aire Bridge: 'A' series specimen (cast 6 May 1965, creep specimens loaded at 7 days).
Creep, shrinkage and elastic strain in concrete bridges

Figure 19: Movement in shrinkage unit and companion specimens on outside exposure, Mancunian Way.

Figure 5: Movements in cantilever of Medway Bridge.
Fig. 3--Experimental creep and shrinkage strains

Fig. 4--At the bridge experimental strains--Average of results from three mixes
Fig. 3 (continued)—Experimental creep and shrinkage strains
TABLE 3 - TEMPERATURES AND HUMIDITIES AT TEST LOCATIONS

<table>
<thead>
<tr>
<th></th>
<th>Average temperature</th>
<th>Average relative humidity</th>
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<tr>
<td></td>
<td>°C</td>
<td>%</td>
</tr>
<tr>
<td>Fog room</td>
<td>22</td>
<td>71.6</td>
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<tr>
<td>Constant room</td>
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</tr>
<tr>
<td>February</td>
<td>20</td>
<td>68</td>
</tr>
<tr>
<td>May</td>
<td>20</td>
<td>68</td>
</tr>
<tr>
<td>August</td>
<td>20</td>
<td>68</td>
</tr>
<tr>
<td>November</td>
<td>20</td>
<td>68</td>
</tr>
<tr>
<td>Year</td>
<td>20</td>
<td>68</td>
</tr>
<tr>
<td>Inside bridge</td>
<td></td>
<td></td>
</tr>
<tr>
<td>February</td>
<td>25</td>
<td>77</td>
</tr>
<tr>
<td>May</td>
<td>20</td>
<td>68</td>
</tr>
<tr>
<td>August</td>
<td>17</td>
<td>67.6</td>
</tr>
<tr>
<td>November</td>
<td>22</td>
<td>71.6</td>
</tr>
<tr>
<td>Year</td>
<td>21</td>
<td>69.8</td>
</tr>
</tbody>
</table>

TABLE 4 - PREDICTED CREEP AND SHRINKAGE FACTORS

<table>
<thead>
<tr>
<th>Compressive of concrete</th>
<th>Current content (lb/sq in)</th>
<th>Air content (0.02)</th>
<th>Aggregate fines (0.2)</th>
<th>Density of factors</th>
<th>Creep at 100 days</th>
<th>Shrinkage at 100 days</th>
<th>Creep at 300 days</th>
<th>Shrinkage at 300 days</th>
<th>Creep at 500 days</th>
<th>Shrinkage at 500 days</th>
<th>Creep at 700 days</th>
<th>Shrinkage at 700 days</th>
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</thead>
<tbody>
<tr>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

**NOTE:** Creep strain = product of creep factors & environmental basic strain

**TABLE 5 - COMPARISON OF EXPERIMENTAL AND PREDICTED STRAINS AT 400 DAYS**

<table>
<thead>
<tr>
<th>Relative humidity %</th>
<th>CEB-PIF prediction</th>
<th>ACI prediction</th>
<th>Experiment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial strain</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Creep strain 100 (fog room)</td>
<td>1.02</td>
<td>1.17</td>
<td>1.20</td>
</tr>
<tr>
<td>80 (inside bridge)</td>
<td>1.98</td>
<td>1.55</td>
<td>1.61</td>
</tr>
<tr>
<td>67 (in constant temperature room)</td>
<td>2.44</td>
<td>1.61</td>
<td>2.82</td>
</tr>
<tr>
<td>Shrinkage strain 100</td>
<td>0</td>
<td>0</td>
<td>-0.40</td>
</tr>
<tr>
<td>80</td>
<td>0.88</td>
<td>2.10</td>
<td>1.92</td>
</tr>
<tr>
<td>Shrinkage strain 67</td>
<td>1.78</td>
<td>3.35</td>
<td>2.03</td>
</tr>
<tr>
<td>Total strain 100</td>
<td>2.02</td>
<td>2.17</td>
<td>1.88</td>
</tr>
<tr>
<td>80</td>
<td>3.86</td>
<td>4.46</td>
<td>3.33</td>
</tr>
<tr>
<td>Shrinkage strain 67</td>
<td>4.72</td>
<td>5.16</td>
<td>5.85</td>
</tr>
</tbody>
</table>

**TABLE 6 - RATE OF DEVELOPMENT OF CREEP AND SHRINKAGE STRAIN**

<table>
<thead>
<tr>
<th></th>
<th>CEB-PIF</th>
<th>ACI</th>
<th>Experiment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Creep strain at 100 days</td>
<td>0.76</td>
<td>0.76</td>
<td>0.67</td>
</tr>
<tr>
<td>Creep strain at 400 days</td>
<td>1.03</td>
<td>1.05</td>
<td>1.05</td>
</tr>
<tr>
<td>Creep strain at 600 days</td>
<td>0.76</td>
<td>0.77</td>
<td>0.76</td>
</tr>
<tr>
<td>Creep strain at 800 days</td>
<td>1.05</td>
<td>1.02</td>
<td>0.91</td>
</tr>
<tr>
<td>Shrinkage strain at 100 days</td>
<td>0.76</td>
<td>0.77</td>
<td>0.76</td>
</tr>
<tr>
<td>Shrinkage strain at 400 days</td>
<td>0.76</td>
<td>0.77</td>
<td>0.76</td>
</tr>
<tr>
<td>Shrinkage strain at 600 days</td>
<td>1.05</td>
<td>1.02</td>
<td>0.91</td>
</tr>
</tbody>
</table>

**Fig. 1--Details of creep and shrinkage specimens**

**Fig. 2(a)--Creep and shrinkage specimens on top of the bridge**
TABLE 1 - RATE OF DEVELOPMENT OF CREEP AND SHRINKAGE STRAIN

<table>
<thead>
<tr>
<th>Time (days)</th>
<th>Creep Strain</th>
<th>Shrinkage Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>28</td>
<td>0.05</td>
<td>0.25</td>
</tr>
<tr>
<td>56</td>
<td>0.10</td>
<td>0.50</td>
</tr>
<tr>
<td>84</td>
<td>0.15</td>
<td>0.75</td>
</tr>
<tr>
<td>112</td>
<td>0.20</td>
<td>1.00</td>
</tr>
<tr>
<td>140</td>
<td>0.25</td>
<td>1.25</td>
</tr>
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</table>

TABLE 3 - COMPARISON OF EXPERIMENTAL AND PREDICTED STRESSES AT 400 DAYS

<table>
<thead>
<tr>
<th>Material</th>
<th>Creep Factor</th>
<th>Shrinkage Factor</th>
</tr>
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<tbody>
<tr>
<td>CEB-FIP</td>
<td>1.00</td>
<td>0.99</td>
</tr>
<tr>
<td>ACI</td>
<td>1.01</td>
<td>0.97</td>
</tr>
</tbody>
</table>

TABLE 4 - PREDICTED CREEP AND SHRINKAGE FACTORS

<table>
<thead>
<tr>
<th>Compo</th>
<th>Creep Factor</th>
<th>Shrinkage Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>28</td>
<td>0.05</td>
<td>0.25</td>
</tr>
<tr>
<td>56</td>
<td>0.10</td>
<td>0.50</td>
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<tr>
<td>84</td>
<td>0.15</td>
<td>0.75</td>
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<td>0.20</td>
<td>1.00</td>
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<tr>
<td>140</td>
<td>0.25</td>
<td>1.25</td>
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Fig. 1 - Details of creep and shrinkage specimens on top of the bridge.
Fig. 2(a) - Creep and shrinkage strains at various ages.
Fig. 3 - Comparison of predicted and experimental creep and shrinkage strains.
<table>
<thead>
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<th>Ref No.</th>
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<tr>
<td>12.7</td>
<td>U. FINSTERWALDER : &quot;Ergebnisse von Kriech und Schwindmessungen an Spannbetonbauwerken&quot; (a) &quot;Beton und Stahlbetonbau&quot; Vol 50-No 2 Januar 1955 pp 44-50 (b) Vol 53-No 5 Mai 1958 pp 136-144</td>
</tr>
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<td>12.8</td>
<td>L.J. PARROTT : &quot;A study of some long-term strains measured in two concrete structures&quot; &quot;Proc Rilem Symposium on testing in-situ of Concrete Structures&quot; pp 123-139.</td>
</tr>
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<td>12.9</td>
<td>L.J. PARROTT : &quot;Long term deformation of Concrete in a Prestressed Concrete Floor&quot; &quot;Proc Glasgow University Conference on Performance of building Structures&quot; March/April 1976 pp 337-351.</td>
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<td>12.15</td>
<td>R.G. TYLER: &quot;Determining stress in concrete structures - Site Method used in precast unit for Western Avenue Extension&quot; <em>Civil Engineering and Public Works Review</em> June 1969 pp 588-596</td>
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<td>R.G. TYLER: &quot;Determining stress in concrete structures - further notes on the method of identical strains&quot; (a) <em>Civil Engineering and Public Works Review</em> June 1971 pp 615-620 (b) &quot; &quot; &quot; July 1971 pp 761-764</td>
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12.38 | D.W. HOBBS: "Influence of specimen geometry upon weight change and shrinkage of air-dried concrete specimens"
12.40 | T.C. HANSEN and A.H. NUTTICK: "Influence of size and shape of Member on the Shrinkage and Creep of Concrete" (a) *American Concrete Institute* Feb 1966 pp 267-289 (b) Discussion: P. Chimavit, K.H. Gerstteg and W.C. Muller Sept 1966 pp 1017-1026.
12.47 | International Association for Bridge and Structural Engineering "Symposium - Design of Concrete Structures for creep, shrinkage and temperature changes" Madrid 1970 (a) Preliminary Publication (b) Final Report
Ref No.  AUTHOR / TITLE / PUBLICATION
12.48  A.S.T.M. Committee C-9: "Tentative Method of Test for Creep of Concrete in compression"  
(a) ASTM Designation: C512-66T  
(b) ASTM C512-76
12.50  British Standard B.S. 1881: "Methods of Testing Concrete"  
(a) Part 3 - Methods of making and curing test Specimen  
(b) Part 4 - Methods of testing Concrete for Strength  
(c) Part 5 - Methods of testing hardened Concrete for other than Strength  
British Standards Institution.
12.51  South African Bureau of Standards - SABS 1083-1976  
12.52  British Standard CP115: 1959  
"The Structural use of prestressed concrete in buildings" British Standards Institution.
12.53  Nasionale Bouvavorsings Instituut - Windhoek August 1980 - vertroulike  
"Voorlopige Tussentydse verslag oor Brugmtings Brakwater en Gross Barman Brue-
12.54  M. EMERSON: "Bridge Temperatures for setting bearings and expansion joints"  
"Transport and Road Research Laboratory" 1979.  
TRRL Supplementary Report 475 pp 1-8.
12.55  M. EMERSON: "Temperature differences in bridges; basis of Design requirements"  
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TRRL Laboratory Report 765 pp 1-16.
12.56  F. LEONHARDT: "Riss-schäden an Betonbrücken-Ursachen und Abhilfe"  
12.57  DR F.S. FULTON: "Concrete Technology - A South African Handbook"  
12.58  B.J. MACKENZIE: "Creep in Concrete - CE506: Properties of Concrete"  
University of Cape Town 1981.
12.59  D. PETERS: "Admixtures - CE506: Properties of Concrete"  
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12.60  GERMAN STANDARD DIN 4227: "Spannbeton, Richtlinien für Bemessung und  
ausführung"  
(a) 1953 & (b) 1979 German Standards Inst.
12.61  F. LEONHARDT: "Spannbeton für die Praxis - 3rd Edition" Wilhelm Ernst & Sohn
12.62  JOSEF AICHHORN: "An Bauwerken Durchgeführte Kriech & Schwind messungen einschliesslich Auswertung zur Bestimmung der Kriechzahlen und Schwindmasse"  
"Proc. International Association for Bridge and Structural Engineering Symp"  
12.63  O.H. MÜLLER: "Some Aspects of the Effect of Micaceous Sand on Concrete"  
The Civil Engineer in SA - September 1971 pp 1-3.
12.64  A.M. NEVILLE: "Creep of Concrete: Plain, Reinforced and Prestressed"  
12.65  Handbook on the Unified Code for Structural Concrete (CP110: 1972)  
Concrete and Concrete Association.
12.66  F. LEONHARDT:  
(a) "Vorlesungen über Massivbau" - Vol 1 "Grundlagen zur Bemessung in  
Stahlbetonbau"  
(b) "Vorlesungen über Massivbau" - Vol 5 "Spannbeton"
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The following courses were successfully completed for 30 Credits towards the M.Sc Degree.

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<td>1971</td>
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<td>Matrix Analysis of Frames Structures</td>
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<td>Surface Structures</td>
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<td>Earth Dam Design</td>
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<td>1980</td>
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<td>Bridge Engineering</td>
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Total = 30 Credits
1. Give a carefully considered account of all the factors involved in the selection of the most suitable method of structural analysis, i.e. FORCE or DISPLACEMENT method, when analysing a particular structure.

[20 marks]

2. For each of the two structures shown below, determine the degree of static and of kinematic indeterminateness; select the method of analysis, and compute the $B_0$, $B_1$, $F_m$ or $K_t$ matrices required for solution. Then indicate clearly the remaining steps in the solution of all member end forces, but do not perform any of the remaining arithmetic processes involved.

(a) Plane Structure:

The cable stayed bridge has a two-span continuous beam deck, which is simply-supported on rollers at the abutments and fixed on top of the rigid pier. The single tower, situated in the centre of the deck width, is rigidly fixed to the deck and the pier. The four similar stay cables are individually clamped to the tower.

The deck has a uniform dead load of 10 k/ft and is subject to a uniformly distributed vertical live loading of 3.5 k/ft of varying length, and a single horizontal braking load of 60 kips applied at the level of the carriageway surface, acting to the left or to the right.

Section properties: Cable $EA = 4 \times 10^6$ k,
Deck $EA = 80 \times 10^6$ k, $EI = 1800 \times 10^6$ k.ft$^2$
Tower $EA = 30 \times 10^6$ k, $EI = 750 \times 10^6$ k.ft$^2$

[35 marks]
Space Structure:

The symmetrical pile foundation of a bridge pier is situated in a river bed which is subject to severe scouring. The reinforced concrete piles are steel-cased and are driven to refusal through 40 ft of loose sand and silt, followed by 60 ft of dense gravel, down to rock. All raking piles are driven at 1:5 to the vertical. The massive pile cap may be assumed infinitely rigid in all respects, and it is subject to a number of forces and moments acting at the level of its underside as shown.

Consider the condition of the river bed scavoured to a depth of 40 ft, assuming that the dense gravel offers complete restraint against horizontal and bending movements, but has a negligible amount of skin friction. The toe of the pile is assumed rigidly fixed against all possible movements.

File section properties:

\[
\begin{align*}
EA &= 1.8 \times 10^6 \text{ kN} \\
EI &= 0.6 \times 10^6 \text{ kN.m}^2 \\
GJ &= 0.5 \times 10^6 \text{ kN.m}^2
\end{align*}
\]

[45 marks]
1. (a) Using the varying grid shown below, write down the finite differences equations for the following nodes: (1), (4), (5) and (8). Then explain the method of obtaining deflections and bending moments at each node, using symbols only. The slab is subjected to a uniform loading throughout as well as a column load at node (8).

![Diagram of slab with nodes and grid]

1. (b) Describe briefly the steps required in the finite element analysis of the shear wall shown.

![Diagram of shear wall with uniform loading and built-in boundary]
2. Derive the finite differences operator patterns to be used at nodes (1), (3), (5), (10), (13) and (18) for the analysis of the raft foundation shown.

Note that a rectangular grid is required. Let $K$ be the foundation modulus and assume that this value is a constant. The raft is subjected to column loads at the positions indicated.

\[
\begin{array}{cccccc}
1 & 2 & 3 & 4 & 5 \\
6 & 7 & 8 & 9 & 10 \\
11 & 12 & 13 & 14 & 15 \\
16 & 17 & 18 & 19 & 20 \\
21 & 22 & 23 & & & \\
24 & 25 & 26 & & & \\
27 & 28 & 29 & & & \\
30 & 31 & 32 & & & \\
\end{array}
\]

All free edges

3. A folded plate type roof is to be constructed so that it consists of four flat inclined surfaces, each supported on a rubber bearing pad at one corner. Each surface will not be quite square, but for such a small roof pitch, a very close approximation to the correct solution can be obtained by using square plate elements.

Show the necessary steps required for a finite element solution of the complete structure subjected to uniformly distributed vertical load. Each surface is to be divided into four square elements.

Show in detail how the stiffness submatrix relating the forces at node (5) to the displacements at node (5) is evaluated. The complete load vector must be given.

Indicate how the stresses and moments are then to be solved at node (5).

rubber pad

\[ a = 4 \text{ metres} \]

\[ v = 0.2 \]
1. With the aid of sketches, briefly describe the effect of filters and dam geometry on the stability of an earth or rockfill dam, and also indicate approximately the types of flow net pattern which can be expected for the different types of dams.

About one third of your discussion should be concerned with the stability aspects. (Suggested length of answer - about four or five pages of writing and sketches.)

[25 marks]

2. Briefly describe one of the following:

(a) The development and dissipation of excess pore water pressure due to dam construction and the effects of this excess pore pressure on the stability of a clay core earth dam. (Illustrate your answer with sketches wherever necessary.)

(b) The various stages in the planning of an earth dam project for irrigation, or municipal water supply purposes. (Suggested length of answer - about three or four pages.)

[20 marks]

3. Briefly, with the aid of sketches, describe one of the following topics: (Answer to be about two or three pages.)

(a) Membranes for earth and rockfill dams.
(b) The use of instrumentation in earth or rockfill dams.
(c) Hydraulic-fill dams.
(d) Floods and hydrology in relation to earth dam design.
(e) Floods and spillways.
(f) Properties and the testing of rockfill material.
(g) The properties, and the testing of materials for earth dams.
(h) Field and laboratory permeability tests.
(i) Site investigations for earth dams.
(k) Practical aspects relating to grouting, membranes, filters and other design details for earth dams.

[15 marks]
4. The inner core of the dam shown in Figure 4 consists of material A with a coefficient of permeability:

\[ k_A = 2 \times 10^{-4} \text{ cm/s} \]

whereas the outer two zones consist of material B which has a coefficient of permeability \( k_B \) which is twice the value of \( k_A \).

(a) Draw a flow net for the dam section.

(b) Briefly list the various construction rules or constraints which should apply to this flow net.

(c) Estimate the rate of seepage through the dam.

5. Consider a typical slice used in the method of slices for slope stability analyses. Draw all the forces which act on this slice and also draw the force polygon. Label and identify all the forces in the force polygon. Hence explain in words (and with the aid of sketches where necessary) the basic assumptions within, and the differences between, different methods of analysis.

For example the following methods could be considered:

(a) the conventional simple method of slices;

(b) the Bishop method (described in Guthrie Brown's Book)

(c) the modified iterative method (Janbu and Bishop) mentioned in the 1967 edition of Terzaghi and Peck's book.

(d) the method for non-circular composite surfaces (Terzaghi and Peck).

Particular attention must be given to the concepts involved. Formulae can assist your discussion but are not essential, unless in a simple form.

6. (a) Briefly describe the method for plotting flow lines and equipotential lines from readings taken during a laboratory test on a model dam, in which a capillary seepage zone exists above the phreatic surface. How can one find the position of the phreatic surface in such a model?

(b) Briefly provide a reason or justification for the use of \( \phi = 0 \) in certain soil stability analyses.
7. A copy of pages 248 to 253 from Terzaghi and Peck's book is supplied for your assistance. Hence perform at least one cycle of calculations to determine the factor of safety \( F \) for the composite sliding surface shown in Figure 7. Use the slices which have been numbered in the sketch. If simplifying assumptions are made, list these above your table. Also list the formula which you have used. For your first try assume \( F = 2.0 \). Suggest the value of \( F \) to use in the second cycle.

A slip surface of the type shown in Figure 7 would be possible if the slope consisted of two different soils. The values of \( c \) and \( \phi \) for the two soils are shown in Figure 7.

The pore pressures may be assumed to be zero. Both soils have a specific weight of 20 kN/m\(^2\).

[35 marks]
SECTION A  COMPULSORY (Both questions to be answered)

A1. Discuss in detail tests of statistical hypotheses describing as many of the test criteria you are aware of, clearly indicating the conditions under which these test criteria are applicable.  (30)

A2. Statistics can be divided into two broad classifications:
(a) Descriptive Statistics
(b) Inferential Statistics.

Describe the various graphical techniques and numerical measures which can be included under the heading of descriptive statistics. The numerical measures must be discussed in detail.  (20)

SECTION B  Answer 3 questions from this section

B1. The resistance (in ohms) of each of a lot of 500 units of a certain electrical product is measured and the results of the measurements (observations) are arranged into a series in ascending order. It is then possible to divide the range into a number of classes over which the observations are distributed. A frequency distribution of the observations emerge from this procedure and is represented in tabular form as follows.

<table>
<thead>
<tr>
<th>Resistance (ohms)</th>
<th>Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.0 - 3.2</td>
<td>2</td>
</tr>
<tr>
<td>3.3 - 3.5</td>
<td>18</td>
</tr>
<tr>
<td>3.6 - 3.8</td>
<td>48</td>
</tr>
<tr>
<td>3.9 - 4.1</td>
<td>97</td>
</tr>
<tr>
<td>4.2 - 4.4</td>
<td>138</td>
</tr>
<tr>
<td>4.5 - 4.7</td>
<td>104</td>
</tr>
<tr>
<td>4.8 - 5.0</td>
<td>69</td>
</tr>
<tr>
<td>5.1 - 5.3</td>
<td>20</td>
</tr>
<tr>
<td>5.4 - 5.6</td>
<td>4</td>
</tr>
</tbody>
</table>

A descriptive analysis of the above data is required.  (20)

B2. Somebody wanted ..........
B2. (a) Somebody wanted to buy a large quantity of light bulbs and had a choice between brands I and II. He bought 100 bulbs of each brand and found by testing that brand I had a mean lifetime of 1120 hours with standard deviation of 75 hours and brand II had a mean lifetime of 1064 with a standard deviation of 82 hours. (Assume that the population variances are equal.) Is the difference in mean lifetimes significant?

(b) Choosing a smaller sample size, we would obtain less information. For what size $n$ would the hypothesis still be accepted when $\alpha = 5\%$ and means and standard deviations are as in (a) above.

(c) Construct two simple samples, so that the hypothesis of equal population means is accepted although the differences between sample means is large. Discuss this. (18)

B3. In a study of the effectiveness of working methods, a firm wanted to compare two types of hand-packing methods. Nine participants in the test obtained the following results (numbers of speckled beans hand-sorted from white and speckled beans within a given interval of time).

<table>
<thead>
<tr>
<th>Work number</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>First method</td>
<td>214</td>
<td>253</td>
<td>276</td>
<td>215</td>
<td>238</td>
<td>221</td>
<td>210</td>
<td>229</td>
<td>269</td>
</tr>
<tr>
<td>Second method</td>
<td>281</td>
<td>279</td>
<td>260</td>
<td>230</td>
<td>267</td>
<td>253</td>
<td>205</td>
<td>265</td>
<td>299</td>
</tr>
</tbody>
</table>

Test the hypothesis that both methods yield equal results against the alternative that the second method is better. (12)

B4. Test for normality of the population from which the following sample ($x =$ tensile strength in kilograms/millimeter$^2$ of steel sheets of 0.3 mm thickness) was taken.

<table>
<thead>
<tr>
<th>$x$</th>
<th>Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>-∞</td>
<td>15</td>
</tr>
<tr>
<td>42,0</td>
<td>11</td>
</tr>
<tr>
<td>42,5</td>
<td>15</td>
</tr>
<tr>
<td>43,0</td>
<td>14</td>
</tr>
<tr>
<td>43,5</td>
<td>23</td>
</tr>
<tr>
<td>44,0</td>
<td>20</td>
</tr>
<tr>
<td>44,5</td>
<td>12</td>
</tr>
<tr>
<td>45,0</td>
<td>19</td>
</tr>
</tbody>
</table>

B5. Graphical Statistics

A contractor is required to provide concrete of a quality such that not more than 10% of the strength tests shall fall below 21 MPa. Control is average, about 5 MPa standard deviation is to be expected.

B5. (a) To what.
(a) To what compressive strength must the contractor design his mix?

(b) Over a period of 10 days his daily test strengths were as follows (ranked):

19,0; 24,0; 2,50; 28,0; 29,5; 30,5; 32,5; 34,0; 37,5; 40,0.

What is the average and standard deviation of the set?

(c) The contractor now receives a different brand of cement and the next set of 10 tests gives the following ranked values:

26,0; 28,0; 31,0; 32,0; 33,5; 35,0; 36,0; 37,5; 39,5; 43,0.

Is there a significant difference between the means of (b) and (c)?

(d) The compression testing machine cylinder develops a leak and cannot raise the stress on a 150 mm cube above 35 MPa. A test on 10 cubes gives the following stress values in MPa:

> 35; 22,5; 34,0; > 35; 26,5; 32,0; > 35; > 35; 28,0; 30,5.

What is the estimated mean strength and standard deviation of the set?
The diagram above shows a cross-section through part of a composite prestressed concrete floor to be built in an area with a mean relative humidity of 50%. For various reasons the precast prestressed beams will be approximately 100 days old when the in-situ slab is added.

(a) Using the CEB-FIP coefficients for shrinkage estimate the eventual differential shrinkage strain of the in-situ slab with respect to the precast beam.

(b) Assuming that the ultimate unrestrained creep strain (i.e. in addition to the above shrinkage strains) in the precast concrete from the time of casting the slab is zero in the top fibre and $400 \times 10^{-6}$ in the bottom fibre, determine the stresses throughout the composite section due to differential shrinkage and creep. Assume linear distribution of strain across the composite section.

(c) Comment on the validity of your findings.

Necessary data:

Precast concrete: Cement content = 500 kg/m\(^3\)
\[ \text{W/C ratio} = 0.40 \]
% longitudinal reinforcement = 1.25%
\[ E \text{ (at 28 days)} = 35 \text{ GPa} \]

In-situ concrete slab: Cement content = 350 kg/m\(^3\)
\[ \text{W/C ratio} = 0.6 \]
% longitudinal reinforcement = 0.25%
\[ E \text{ (at 28 days)} = 28 \text{ GPa} \]
The symmetrical simply-supported post-tensioned prestressed concrete I-beam shown above carries the timber floor of a sports centre. After construction of the beam and application of the full prestressing force, walls and windows are erected at A and E giving a permanent load of 20 kN at each of these points. The timber floor is then constructed imposing a load of 1 kN/m along the full length of the beam. The floor is required to carry an imposed load equivalent to 15 kN/m on any length of the beam (All the above loads are characteristic values). The weight of the concrete may be taken to be 25 kN/m³ and the additional concrete forming the "end blocks" of the beam may be neglected.

Determine the following:

(a) the minimum prestressing force necessary for the beam to comply with CP 110 as a Class 3 member. (Permissible stresses and other data are given below). Assume a uniform prestressing force (i.e. friction losses neglected);

(b) an approximate tendon profile for the beam giving ordinates at A, B and C only;

(c) the area of additional untensioned high yield reinforcement required at midspan to give adequate ultimate strength in accordance with CP 110. (To be calculated without the use of Design charts. Design curves for prestressing wire and high yield steel are attached).

In what way, if any, could the situation be improved by means of a cable transformation?
Necessary data:

Use Concrete Grade 50 (i.e. $f_{cu} = 50$ MPa)
Permissible compressive stress for serviceability limit state: 16.7 MPa

Permissible hypothetical tensile stress for serviceability limit state: $5.8 \times 0.8 \times 0.8$ Depth factor $= 4.64$ MPa.

Minimum vertical cover distance to centroid of prestressing tendons: 100 mm

Factors of Safety for Ultimate limit state: Dead Load: 1.4
Imposed Load: 1.6

Characteristic strength of prestressing wire: $f_{pu} = 1550$ MPa
Residual prestress (after losses) $= 0.6 f_{pu}$

Characteristic strength of high yield steel: $f_y = 410$ MPa

Minimum concrete cover to longitudinal reinforcement: 20 mm

[45 marks]
1. Write brief notes of about one page, on each of the following topics:
   
   - situations where steelwork is particularly suitable for bridge superstructures in South Africa;
   - effects of the introduction of railways on the evolution of bridge engineering;
   - wind loading on bridges;
   - standard precast concrete bridge beams;
   - steel box girders or concrete box girders;
   - load combinations for highway bridges;
   - differences between solid, voided and cellular slab bridge decks.

   (50 marks)

2. A flyover road bridge of conventional beam and slab construction has two 20 m spans and is 12 m wide between kerbs (4 lanes). There are 5 longitudinal beams and a crossbeam at the middle of each span. The superstructure is continuous across the central pier and is simply-supported on closed abutments. There is a bearing under each longitudinal beam at every support and the bridge is not skew in plan.

   Determine the intensity, position and extent of the "normal" (HA or NA) loading to be applied for the following effects:

   (a)/................
maximum sagging moment at midspan in an edge beam;
maximum hogging moment in a crossbeam at midspan;
maximum hogging moment over pier in the middle longitudinal beam;
minimum reaction on the middle bearing on an abutment;
maximum reaction on the edge bearing on the pier.

Use either BS 153 (1972) or BS 5400 (1978), or CPA (1977), or NTC (1977) road traffic specification.

(20 marks)

3. For each of the two bridge sites on the attached sheets, select the most suitable type of bridge structure and construction method. Draw adequate sketches of the superstructure, substructure and foundations directly on these sheets and hand them in. State all assumptions clearly and describe the construction method adequately. List brief reasons for all the major decisions.

(15 marks each)