THE INFLUENCE OF AGGREGATE STIFFNESS ON THE MEASURED AND PREDICTED CREEP BEHAVIOUR OF CONCRETE

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A project report submitted to the Faculty of Engineering, University of the Witwatersrand, Johannesburg, in partial fulfilment of the requirements for the degree of Master of Science in Engineering.

Johannesburg, 1998

DECLARATION

I, George C Fanourakis, hereby declare that this project report is my own, unaided work. It is being submitted for the degree of Master of Science in Engineering in the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other University.

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Dated this 25th day of August, 1998

ABSTRACT

Aggregate stiffness is known to influence the magnitude of creep of concrete. The purpose of this research project was to quantify the influence of aggregate stiffness on the measured and predicted long-term creep behaviour of plain concrete.

Basic and total creep tests were conducted on concrete specimens of two different strength grades for each of three different commonly used South African aggregate types (quartzite, granite and andesite). In addition, elastic modulus tests were conducted on cores of the aggregate types considered.

The test results revealed that no correlation exists between the creep of concrete and the stiffness of the included aggregate. These results appear to be attributable to the stress-strain behaviour of the aggregate/paste interfacial zone, in the case of aggregates with an elastic modulus in excess of 70 GPa.

The experimental basic and total creep values from this investigation were compared with those predicted for each mix at the corresponding ages by the BS 8110 (1985), ACI 209 (1992), AS 3600 (1988), CEB-FIP (1970), CEB-FIP (1978), CEB-FIP (1990) and the RILEM Model B3 (1995). This comparison indicated that the results predicted by each model vary widely and that no correlation exists between the magnitude of the aggregate stiffness and the creep strains predicted by each model.

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Thanks be to the Almighty God, the source of all strength, guidance, knowledge and wisdom. For all glory, honour and worship belong to You, Father, Son and Holy Spirit, now and forever. Amen.

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CHAPTER 1

INTRODUCTION

1.1 Creep Strain and its Significance

1.1.1 Creep strain

Creep is the time dependent increase in strain of a solid body under constant or controlled stress. Creep deformations are normally two to three times larger than the instantaneous deformations of concrete (Bazant and Panula, 1978).

Creep strain (at any time) can be divided into a basic creep and a drying creep component. If the concrete is sealed or if there is no moisture exchange between the concrete and the ambient medium, only *basic* creep occurs. *Drying* creep is the additional creep experienced when the concrete is allowed to dry while under sustained load. The sum of *basic* and *drying* creep is referred to as *total* creep.

1.1.2 Factors affecting creep

Creep of concrete is affected by many variables. Some of these variables are intrinsic properties of the mix and others are associated with extrinsic environmental factors.

The intrinsic factors include water/cement (w/c) ratio, degree of hydration, age of paste, cement type, admixtures, moisture content, aggregate content and aggregate properties. The extrinsic factors include stress/strength ratio, duration of load, curing, age of loading, member geometry, relative humidity, temperature and rate and time of drying.

With regard to the study of creep, concrete is generally considered as a two-phase material consisting of the hardened cement paste and the aggregate. Creep strain is assumed to occur in the cement paste, provided the aggregate does not exhibit any time dependent deformation of its own. However, when the creep strain in the cement paste under a sustained load increases with time, the actual deformation of the concrete is restrained by the presence of the aggregate which is interspersed in the paste (Neville et. al., 1983).

1.1.3 The effects of creep

Creep of concrete is both a desirable and an undesirable phenomenon. On the one hand it is desirable as it imparts a degree of ductility to the concrete, without which the concrete would be too brittle for use in the majority of structures (Alexander, 1986). On the other hand, creep is often responsible for excessive deflections at service loads which can result in the instability in arch or shell structures, cracking, creep buckling of long columns and loss of prestress (RILEM Model B3, 1995). Frequently the detrimental results of creep are more damaging to non-load-bearing components associated with the structure, such as window frames, cladding panels and partitions, than they are to the structure itself (Davis and Alexander, 1992). Often, damaged structures are either shut down or undergo extensive repairs long before the end of their intended design life, resulting in significant economic consequences. Creep strain is generally associated with its detrimental effects.

Thus the magnitude of creep is a design consideration which is of importance for the durability and the long-term serviceability of structures. Its importance has been heightened by the increasing tendency to use highly stressed and slender members.

1.2 The Determination of Creep Strain

1.2.1 Accuracy of estimations

The magnitude of creep, which is required for design purposes, can be estimated at various levels of accuracy. The choice of level depends on the type of structure and the time of prediction with regards to the information available. Therefore, in cases where only a rough estimate of the creep is required, this estimate can be made on the basis of a few parameters which are available at the design stage, such as characteristic compressive strength of concrete, member thickness and relative humidity. On the other extreme, in the case of deformation-sensitive structures where an accurate estimate is required, an initial estimate of the creep can be made at the design stage using the procedure discussed above. The accuracy of this estimate can subsequently be improved by utilising relatively complex creep prediction models which require specific information on the mix design (such as 28 day compressive strength of the concrete, w/c ratio and binder type) which is available at the early stages of construction. Further refinement of the creep estimation may follow in the case of prestressed structures where post-construction *in-situ* measurements can be used as a basis for adjusting the stresses in the tendons, thereby ensuring that the structure conforms with the design. Ideally, a compromise has to be sought between the simplicity of the prediction procedure and the accuracy of results required.

As long-term testing is impractical and often not an option where accuracy is required, short term tests, performed during a time period not exceeding 28 days after exposure, may be used to adjust the relatively simple models which are incorporated in design codes (code-type models) in order to obtain a more accurate prediction of the long-term creep of concrete (Ojdrovic and Zarghamee, 1996). Alternatively the results of short term creep tests on a given concrete may be extrapolated to long term results by combining the measured data with prior statistical information on the creep of concrete in general, using, for example, the Bayesian statistical approach developed by Bazant and Chern (1984).

1.2.2 Prediction models

(reep predictions for structures that are not deformation-sensitive are usually arrived at by means of the application of generally simple code-type models. These models vary widely in their techniques, are empirically based and do not require any results from laboratory tests as input. However, certain intrinsic and/or extrinsic variables, such as mix proportions, material properties and age of loading are required as input to these models. Such models include the:

- British Standards Institution - Structural Use of Concrete, BS 8110 - Part 2 - (1985)
- American Concrete Institute (ACI) Committee 209 (1992)
- Standards Association of Australia - Australian Standard for Concrete Structures - AS 3600 (1988)

- Comité Euro-International Du Béton - Federation Internationale De La Precontrainte (CEB-FIP) Model Code (1970)
- CEB-FIP Model Code (1978)
- CEB-FIP Model Code (1990)
- International Union of Testing and Research Laboratories for Materials and Structures (RILEM) Model B3 (1995)

1.3 Motivation for this Investigation

Although research on creep has been actively pursued for the past 90 years, significant improvement of the present understanding and prediction capabilities is justified for the following reasons :

- The realistic prediction of creep is difficult as this phenomenon is caused by several interacting mechanisms and is affected by many factors (RILEM Model B3, 1995). For this reason, despite the publication of many empirical creep prediction methods, a unified approach incorporating all relevant factors and which is applicable to all situations is yet to be developed;

- According to the RILEM Model B3 (1995), Brooks et al., (1992), McDonald et al., (1988), Alexander (1986) and Gilbert (1988), estimates of creep obtained from the different code-type predictive methods vary widely. Therefore, despite the research conducted to date, much more experimental work is required before a model is produced which accurately accounts for the many parameters which affect creep;

- The uncertainty in the predictions from code type models may, to some degree, be attributable to the fact that, of the methods listed above, only the BS 8110 (1985) method directly accounts for the influence of aggregate stiffness which is known to effect the magnitude of creep Hobbs (1971);

- The accuracy of existing creep prediction models when applied to concretes containing South African aggregate and binder types and exposed to South African environmental conditions has not been assessed.

Hence, the above needs initiated the investigation project which is described in this dissertation.

1.4 Objectives of this Investigation

The purpose of this investigation was to assess the influence of aggregate stiffness on the long-term (up to six months) creep behaviour of plain concrete and to determine the suitability of a number of existing prediction models.

The specific objectives of this investigation were to:

- Evaluate the effect of different w/c ratios (0.56 and 0.4) on the basic and total creep behaviour of concrete;

- Assess the effect of aggregate stiffness on the basic and total creep behaviour of concretes containing one of three different commonly used South African aggregate types. The aggregate types considered were quartzite from the Ferro quarry in Pretoria, granite from the Jukskei quarry in Midrand and andesite from the Eikenhof quarry in Johannesburg, South Africa;

- Confirm, and possibly supplement, the findings of Davis and Alexander (1992) who conducted research on the total creep of concretes containing either quartzite, granite or andesite from the same sources as those used in this investigation;

- Compare the experimental basic and total creep values from this investigation against those predicted at the corresponding ages by the BS 8110 (1985), ACI 209 (1992), AS 3600 (1988), CEB-FIP (1970), CEB-FIP (1978), CEB-FIP (1990) and the RILEM Model B3 (1995) methods;

- Compare the accuracy of the above mentioned prediction methods established in this investigation with the accuracy determined for the same methods from results of other investigations;

- Identify the method or methods which provide the most accurate estimates of creep strain of concrete made with either quartzite, granite or andesite aggregates;

The ultimate objective of the investigation was to provide designers with information regarding the most suitable method for predicting the creep of concrete comprising any of the aggregates included in the investigation, thereby obviating the need for laboratory creep tests where any of these aggregates are used in concrete.

1.5 Organisation of this Dissertation

Chapter 2 provides a general review of the literature on creep with particular attention on the factors affecting creep, the physical mechanisms of creep and the influence of aggregate stiffness on creep.

Chapter 3 provides an overview of the prediction methods used in this study. The approach and applicability of each method is also discussed.

Details on the materials and equipment used and tests conducted in this investigation are discussed in Chapter 4 and the results obtained are presented and discussed in Chapter 5.

In Chapter 6, the measured strains are compared with the strains predicted at the corresponding time periods by the seven prediction methods, six of which derive from structural codes, included in this investigation. This chapter includes details on any assumptions made in the predictive procedures and statistical techniques employed in evaluating the accuracy of the predicted results. The findings are discussed and compared with those of similar projects carried out by other researchers.

Finally, Chapter 7 summarises the more important findings and conclusions of this study and provides specific recommendations with regards to further research based on these findings.

CHAPTER 2

LITERATURE REVIEW

2.1 The Phenomenon of Creep

2.1.1 Total strain

At any time t, the total concrete strain $\varepsilon(t)$ in a specimen subjected to a uniform sustained uniaxial stress can be expressed by the following equation :

$$\varepsilon(t) = \varepsilon_e(t) + \varepsilon_c(t) + \varepsilon_{sh}(t) + \varepsilon_T(t) \qquad (2.1)$$

where,

$\varepsilon_e(t)$	=	instantaneous elastic strain
$\varepsilon_c(t)$	=	creep strain
$\varepsilon_{sh}(t)$	=	shrinkage strain
$\varepsilon_T(t)$	=	temperature strain (omitted from the equation for the rest of this project on the assumption of constant temperature conditions)

It is usual to assume that the creep and shrinkage components are independent and may be calculated separately and summed to obtain the total non-elastic strain, as shown in Figure 2.1. According to Neville et. al., (1983), Reutz (1965), Kovler (1996), Powers (1966) and Pickett (1942) this principle of superposition is not entirely correct since creep and shrinkage are not independent phenomena. Nevertheless, all available information on creep and its prediction are based on the assumption of the additive effects of creep and shrinkage.

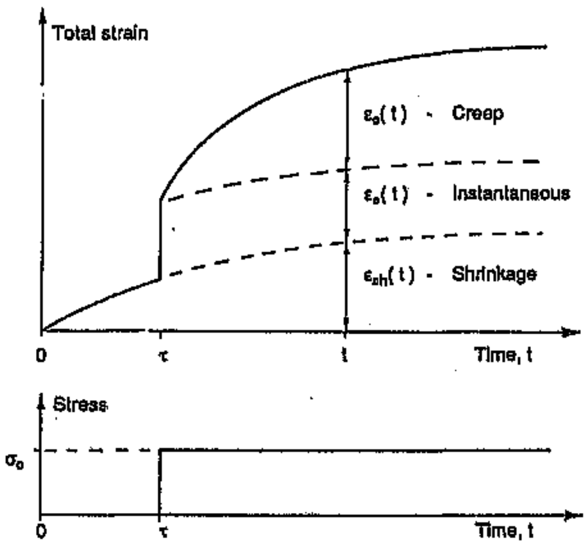


Figure 2.1 Concrete strain components under sustained stress

2.1.2 Creep strain

Total creep strain, $\varepsilon_c(t)$, at any time t, can be divided into several components as follows:

$$\varepsilon_c(t) = \varepsilon_d(t) + \varepsilon_{fi}(t) + \varepsilon_{fb}(t) + \varepsilon_{fd}(t) \tag{2.2}$$

where,

$\varepsilon_d(t)$	=	delayed elastic strain (recoverable)
$\varepsilon_{fi}(t)$	=	rapid initial flow which occurs within 24 hours and is dependent on the age at first loading
$\varepsilon_{fb}(t)$	=	basic flow which depends on the composition of the concrete mix
$\varepsilon_{fd}(t)$	=	drying flow which depends on the moisture content and gradient as well as the size and shape of the specimen

The drying flow creep component is generally referred to as drying creep whereas the remaining three components constitute the basic creep. Drying creep will only take place under conditions where the concrete is allowed to dry while under sustained load. The basic creep will occur when conditions are such that no moisture exchange between the concrete and the ambient medium is permitted. The components of creep are illustrated in Figure 2.2

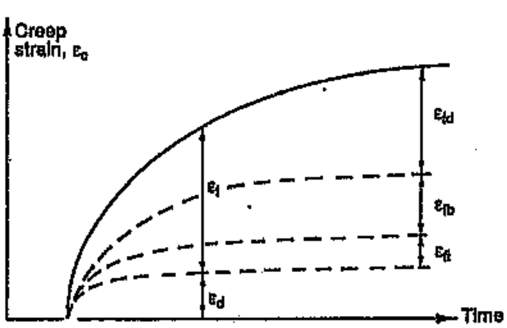


Figure 2.2 Creep components in a drying specimen (after Gilbert, 1988)

The above strain-time curves (Figures 2.1 and 2.2) are representative of situations where the uniaxially applied sustained constant stress does not exceed 40 per cent of the short term strength of the concrete. At higher stress levels the mechanisms are different as significant load induced micro-cracking will occur between the aggregate and the matrix and within the matrix, hence leading to an additional creep component (Alexander, 1994a).

Pickett (1942) introduced the concept of drying creep to account for the difference between the total creep and basic creep. Clearly this concept deviates from the assumption that creep and shrinkage are additive. "This deviation will inevitably arise if the deformation of a drying specimen is expressed through that of a sealed one because these two specimens represent different materials, in spite of the same structure and geometry, age and load conditions, and even the same total moisture content (in the initial moment of sealing). The main difference is that the distribution of moisture and vapour pressure in sealed specimens is changed immediately after sealing. Therefore, the deformations of sealed concretes should proceed according to their own law, as deformations of another material " (Kovler, 1996).

2.1.3 Creep-time functions

The specific creep (Co), creep coefficient (ϕ) and creep function (Φ), as defined below, are generally used for the computation of comparative creep strain.

Specific creep (creep strain per unit applied stress) $C_c = \dfrac{\varepsilon_c}{\sigma}$ (2.3)

Creep coefficient (creep strain divided by initial elastic strain) $\phi = \dfrac{\varepsilon_c}{\varepsilon_e}$ (2.4)

Creep function (sum of instantaneous and creep strains) $\Phi = \dfrac{1}{E_c(\tau)}\left[1 + \phi(t, \tau)\right]$ (2.5)

where: ε_c is creep strain, ε_e is instantaneous elastic strain; σ is the applied constant stress, E_c is the elastic modulus of the concrete; t is the age of the concrete and τ is the time at which the load was applied. Therefore $(t-\tau)$ is the time under load.

2.1.4 Shrinkage strain

Shrinkage is defined as the time-dependent reduction in the volume of fresh or hardened concrete. The shrinkage strain of hardened concrete that will occur at any time t, $\varepsilon_{sh}(t)$, depends on the external environment and occurs in the paste of the concrete (unless shrinking aggregates are used) (Alexander, 1994a). Concrete that is exposed to an environment where drying is permitted will exhibit drying shrinkage whereas concrete that is sealed from the environment (eg. very large members) will exhibit autogenous shrinkage. Concrete that is stored in water will swell (negative shrinkage). Shrinkage due to carbonation may also occur in hardened concrete. Drying shrinkage and autogenous shrinkage, which are of relevance to this project, are briefly described below.

Drying shrinkage

Drying shrinkage is associated with the outflow of moisture from the concrete to the environment, hence resulting in a decrease in the volume of the concrete. This moisture loss occurs as conventional concrete contains more water than can chemically be combined with the cement (Alexander, 1994a).

Autogenous shrinkage

Autogenous shrinkage is often referred to as chemical shrinkage. Since the products of hydration occupy less volume than the sum of the volumes of the original separate components, the cement/water system contracts as hydration proceeds. Furthermore, if no additional water is made available after mixing, the consumption of the water in the concrete during the hydration process results in autogenous shrinkage. The magnitude of the autogenous shrinkage relative to the drying shrinkage is small (Alexander, 1986).

The results of shrinkage tests conducted by Alexander (1994b) on concrete containing either ordinary portland cement (OPC) or rapid hardening portland cement (RHPC) or blended cements (incorporating blastfurnace slag and condensed silica fume) show the magnitudes of the autogenous shrinkage to be significant. This finding is in contradiction with RILEM TC-107 (1995) which states that the magnitude of autogenous shrinkage is usually negligible. Figure 2.3 illustrates the shrinkage exhibited by the sealed and exposed RHPC (mix 2.2) and OPC (mix 2.1) specimens. At present, in South Africa CEM I 42,5 cement is the equivalent of OPC cement. The mix proportions of the abovementioned concretes are given in Table 2.1.

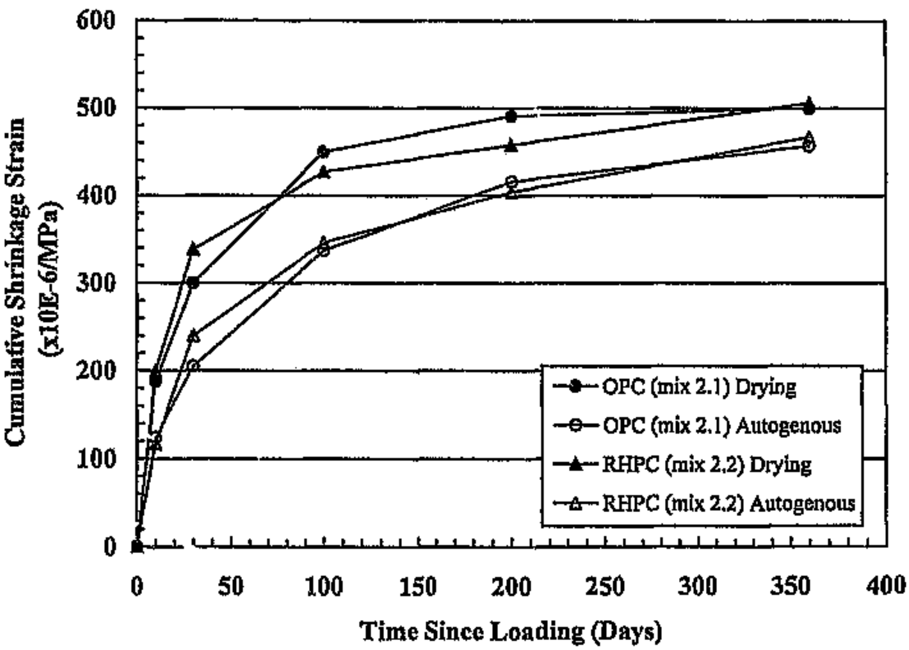


Figure 2.3 Comparison of RHPC (mix 2.2) and OPC (mix 2.1) drying and autogenous shrinkage strains (after Alexander, 1994b)

Table 2.1 Concrete mix proportions of mixes 2.1 and 2.2 (after Alexander, 1994b)

Cement Type	OPC	RHPC
Mix Number	2.1	2.2
Water (l/m^3)	190	190
Cement (kg/m^3)	370	330
19mm Quartzite Crushed Stone (kg/m^3)	1150	1150
Quartzite Crusher Sand (kg/m^3)	560	578
Natural Alluvial Filler Sand (kg/m^3)	185	200
W/C Ratio	0.51	0.57
A/C Ratio	5.12	5.84

The results shown in Figure 2.3 indicate autogenous shrinkage strain magnitudes of approximately 450 micro-strain at an age of 360 days. According to Alexander (1998), these magnitudes are unusually high and, typically, autogenous strains not exceeding 150 micro-strain would be expected at an age of 360 days.

2.2 Factors Affecting Creep of Concrete

Creep of concrete is affected by many variables. Some of these variables are intrinsic properties of the mix or specimen and others are associated with extrinsic environmental factors (Alexander, 1994a). The influence of the different factors on creep is discussed by Alexander (1994a) and others and is briefly summarised, for the uniaxial compressive stress state, below.

2.2.1 Intrinsic factors

Water : cement ratio

A decrease in the w/c ratio of a mix results in an increase in the strength and stiffness and a decrease in the permeability of the cement paste. Hence, a decrease in the w/c ratio causes a decrease in creep. This is shown in Figure 2.4, which has been adapted by Alexander (1994a) from Ruetz (1965).

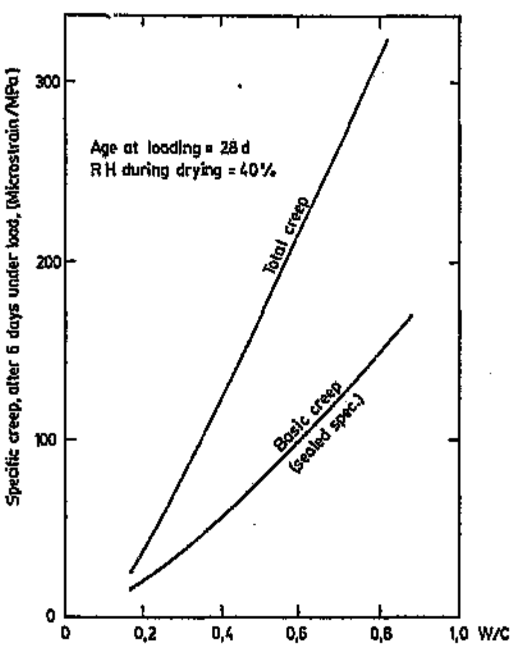


Figure 2.4 Effect of w/c ratio on creep of cement paste (after Ruetz, 1969)

Moisture content

The greater the moisture content of the cement paste at the time of loading and while under load, the greater the creep. This is due to the association of creep with the presence of mobile water in the paste. This relationship is shown in Figure 2.5. This figure also illustrates that the greater the extent to which pastes or concretes are allowed to dry before loading, the less the magnitude of the creep. Therefore, concretes that are first subjected to drying at loading will exhibit more creep than if drying prior to loading was permitted (Wittmann, 1970).

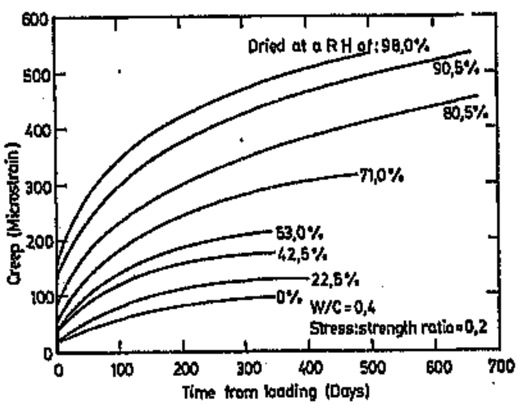


Figure 2.5 Effect of previous drying on basic creep of cement paste (after Wittmann, 1970)

Cement type

The creep of cement paste is affected by the cement composition as follows:

- Cements with higher C_3A or lower C_3S contents exhibit relatively higher creep strains (Mindess and Young, 1981)

- Creep increases in magnitude with the use of the following cements: rapid-hardening, ordinary portland and low heat (Neville et. al., 1983), (Alexander, 1994a). According to Alexander (1994a), at least a difference of the portion of the creep resulting from the use of different cements can be accounted for by the variable strength gain rates which are affected by the composition and fineness of grinding.

Cement extenders

Work by Alexander (1994b) showed that the presence of ground granulated blastfurnace slag (GGBS) in ordinary portland cement (OPC) concretes caused a possible small increase in creep (approximately 20 per cent) at early ages in drying specimens, but that this effect was usually reversed at lager ages.

The use of fly ash (FA) in portland cement concretes had the effect of reducing the specific creep in comparison with plain mixes with similar 28 day strengths, particularly in the case of sealed specimens (Grieve, 1991), (CSIR, 1982), (Dhir et. al., 1986) and (Carette and Malhotra, 1986).

Limited research on the use of condensed silica fume (CSF) and OPC in concrete indicated a reduction of the creep magnitude when compared with otherwise similar OPC mixes (Alexander, 1994b), (Luther and Hansen, 1989), (Buil and Acker, 1985) and (Wolsiefer, 1982).

Admixtures

The effect of admixtures on creep has been found to be highly variable depending on the composition and type of admixture used (Morgan, 1975), (Alexander, 1983) and (Brooks, 1989). Therefore, when the use of an admixture is proposed for concrete where creep may detrimentally affect the structural performance, it is recommended that the effect of the admixture on creep should be assessed by means of laboratory tests (Alexander, 1994a).

Aggregate properties and content

Normal-density aggregates of crushed rock or hard gravel normally do not exhibit creep at the stress levels to which they are subjected in normal concrete. Hence, aggregates reduce the creep of concrete by diluting the paste and restraining its movement. The particle shape, maximum size and grading of the aggregate are an important factor as they influence the volume fraction of aggregate in the concrete (Ballim, 1983). From the above it is clear that creep of concrete is affected by both the aggregate volume concentration and the stiffness of the aggregate (Alexander, 1994a). In addition, the profound influence of aggregate type on the deformation properties of concrete was proved by Alexander (1996). A more detailed discussion on the influence of aggregate stiffness on creep is given in Section 2.5.

2.2.2 Extrinsic factors

Member geometry and size

The shape and size of a member are reflected in the volume-to-surface ratio (Hansen and Mattock, 1966). In the case of drying under load, the larger the volume-to-surface ratio the longer the diffusion paths for moisture loss, hence the lower the early rate of drying creep and the ultimate creep (CEB-FIP, 1978). This is shown in Figure 2.6.

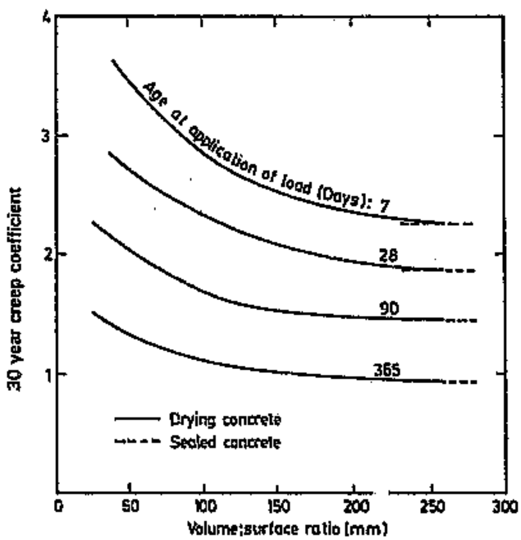


Figure 2.6 Thirty year creep coefficient versus volume: surface ratio for sealed concrete and for drying concrete stored at a relative humidity of 60 per cent (after Neville and Brooks, 1987)

Drying conditions (relative humidity and temperature).

In the case of creep tests where drying is permitted, the total creep (sum of basic and drying creep) will increase with a decrease in ambient relative humidity or an increase in temperature (Alexander, 1994a). This is confirmed by Bhal and Mittal (1996) who found that the ultimate creep at 50 per cent relative humidity is approximately three times the creep at 100 per cent relative humidity. According to Troxell et al., (1958) concretes which are permitted to dry out for the first time under load exhibit considerably higher creep magnitudes at lower relative humidities. This effect is illustrated in Figure 2.7.

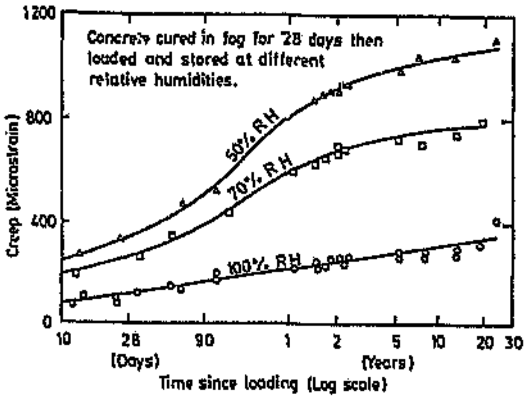


Figure 2.7 Effect of relative humidity on creep of concrete (after Troxell et al., 1958)

The effect of an increase in temperature on concrete creep is dependent on the time at which the temperature rise occurs relative to the time of load application (Bamforth, 1980). According to Neville and Brooks (1987) concretes which are cured at test temperature will exhibit less creep than concretes which are cured at a temperature of 21 °C and then heated to test temperature at one week before loading (refer to Figure 2.8). Note that the specimens were cured (saturated) at the stated temperature from one day until loading at one year.

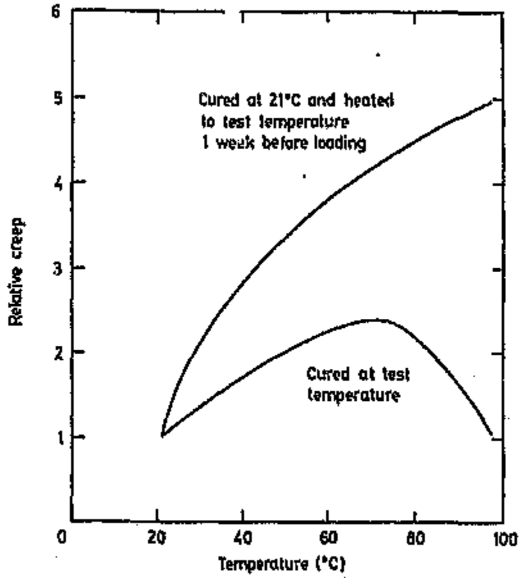


Figure 2.8 Influence of temperature on creep (after Neville and Brooks, 1987)

Stress:strength ratio, curing and age at loading

The stress:strength ratio incorporates the effect of a number of factors which affect concrete creep. These factors include the magnitude of stress, duration and type of curing, age at loading and w/c cement ratio. For constant mix proportions and the same aggregate type, creep increases with higher stress and decreases with increasing strength at the time of load application (Alexander, 1994a). The relationship between stress:strength ratio and creep is assumed to be linear for stress levels less than approximately 40 per cent of the short term strength (Neville et al., 1983). Furthermore, the later the age of load application, the less the expected creep (provided that adequate curing has been achieved). This trend is shown in Figure 2.9 which is based on tests by different investigators (Neville et al., 1983).

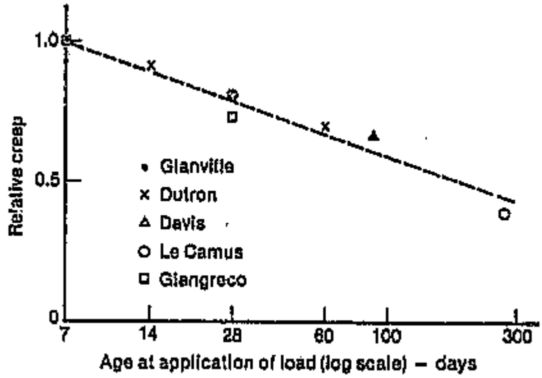


Figure 2.9 Influence of age of load application on creep of concrete relative to creep of concrete loaded at 7 days and stored at a relative humidity of approximately 75 per cent (after Neville et al., 1983)

Time under load

Creep of concrete occurs as long as it is subjected to an external load, if not indefinitely. It is evident from Figure 2.7 that measurable creep still occurs at an age of approximately 30 years after loading. In general, approximately 50 per cent of the 20 year creep occurs within a period of two to six months after loading, while 80 per cent occurs after about one or two years (Alexander, 1994a).

2.3 Mechanisms of Creep

The results of many experiments have led to the development of models to explain the mechanisms of creep. However, a review of the proposed mechanisms indicated that many of these were subsequently discredited by new tests. This is an indication that the mechanisms of creep are not yet clearly understood (Yunping and Jennings, 1992). The broad mechanisms that have been identified are briefly discussed below. These mechanisms are applicable to stresses in concrete not exceeding 40 per cent of the ultimate compressive strength.

2.3.1 Mechanical deformation theory

According to this theory, when concrete is subjected to a compressive stress, the form of the capillary structure in the cement paste changes due to the applied load and resulting internal stresses (Freyssinet, 1936). This entails the deformation of the capillaries and the outward displacement of the water menisci to points where the capillary diameters are larger, resulting in a decrease in the tension under which the capillary water is held. Consequently, the induced compressive stress is reduced. Furthermore, the hygral equilibrium is upset and water will evaporate from the capillaries until the vapour pressure is reduced to the ambient value. The tension in the capillary water rises and, in order to maintain equilibrium, an increase in the compression of the solid phase results. The resultant deformation is the creep (Neville et al., 1983).

This theory describes a delayed elastic type strain, in which case the creep would be reversed if the load were to be removed. As this hypothesis cannot explain creep in water, it is not accepted.

2.3.2 Plastic theory

This theory suggest that the creep of concrete may be in the nature of the crystalline flow. In other words creep occurs as a result of slipping along planes within the crystal lattice, similar to plastic flow of metals. Although this theory may be applicable to concrete subjected to stresses of near failure magnitude, this type of behaviour is not of great significance to the creep behaviour of concrete under normal loads (Neville et al., 1983).

2.3.3 Viscous and visco-elastic flow theory

This theory states that the hydrated cement paste is a highly viscous liquid whose viscosity increases with time due to chemical changes within the structure. This was initially suggested by Thomas (1937) and was reiterated by Reiner (1949).

Hansen (1960) was of the opinion that the viscous flow in hydrated cement paste took place at the grain or particle boundaries.

The viscous flow theory is one of the most important creep theories and there are strong reasons to believe that this flow contributes to the creep of concrete. However, it is not clear whether it is the water or the gel that constitutes this viscous phase (Neville et al., 1983).

2.3.4 Elastic after-effect theory

This theory denies the existence of creep and was expressed by Maney (1941). The basis of this theory is that true creep is not appreciable at working loads and that the effect of loading is simply an elastic change due to the change of non-uniform shrinkage (Neville et al., 1983).

2.3.5 Seepage solution theory

This theory assumes that the hydrated cement paste is a rigid gel and that when a load is applied to the concrete, an expulsion of the viscous components from the voids in the elastic skeleton results. This in turn leads to a redistribution of the stresses from the viscous component to the elastic skeleton. Therefore, the creep is due to the seepage of gel water under pressure. Note that only the gel water is involved in this movement and not the capillary or chemically bound water (Neville et al., 1983). This theory was supported by Lea and Lee (1946) and Seed (1948).

This theory provides an explanation for the large creep exhibited by drying concrete in comparison to that of wet or dry concrete (Yunping and Jennings, 1992).

2.4 Creep Hypotheses

Each of the mechanisms described above may form the basis of a m. ematical model for creep. However, the development of a comprehensive model may result from the combination of one or more of the proposed mechanis. To date no universally accepted mechanism or hypothesis has been established, probably as their is little evidence at a microstructural level to separate one mechanism from another (Yunping and Jennings, 1992).

In view of the fact that not one of the abovementioned mechan' counts for the observed phenomena, integrated theories based on the combination of more than one mechanism have been developed. A number of hypotheses representative of the different schools of thought are briefly described below.

2.4.1 American Concrete Institute (1972)

The American Concrete Institute (ACI) (1972) have attributed creep to the following four main mechanisms:

Viscous flow

This occurs in the cement paste and is caused by sliding or shear of the gel particles which are lubricated by layers of adsorbed water.

Seepage

Consolidation occurs due to seepage of adsorbed water or the decomposition of interlayer hydrate water.

Delayed elasticity

This component accompanies viscous flow and seepage (above) and is due to the cement paste which acts as a restraint on the elastic deformation of the aggregate and gel crystals.

Permanent deformation

This is due to any local fracture, including microcracking and crystal failure, as well as formation of new physical bonds and recrystalization.

According to the ACI (1972) the bulk of creep exhibited by concrete is due to the viscous flow and seepage mechanisms.

2.4.2 Powers' hypothesis

Powers (1966) states that the quasi-crystalline solid bodies comprising the hydrated cement paste, which are mostly colloidal sized, are arranged in such a way that a large proportion of the interstitial spaces are not wide enough to accommodate the number of adsorbed water layers that can be held in the wider spaces at a given relative humidity. This obstruction of adsorption results in a disjoining pressure (Neville et al., 1983).

Although this water is load bearing and represents a structural element of the hardened cement paste it is more mobile than a solid. Therefore, when an external stress is applied to the concrete, the load bearing water in the areas of hindered adsorption is subjected to an additional pressure. To preserve hygral equilibrium, this water diffuses to adjacent areas of unhindered adsorption, reducing the swelling pressure as well as the thickness of the load bearing films, consequently resulting in a reduction in inter-particle spacing. In the case of loaded concrete which is permitted to dry, the water molecules are eventually transferred out of the system.

The reduction is in the direction of the applied load and constitutes the creep. The magnitude of the creep depends on the amount of water in the load-bearing area that must be moved to restore the hygral equilibrium. According to this hypothesis, which is concerned with reversible creep only, creep recovery is the reversal of the above process which occurs when changes occur due to a drop in the pressure of the load bearing water (Powers, 1966).

2.4.3 Ishai's hypothesis

According to Ishai (1968), the application of an external load to a concrete member results in an instantaneous elastic response of the solid phase and of the liquid in the cavities. Hence, the load is carried by the solid and the liquid phases.

Under sustained load the compressed liquid in the cavities diffuses to areas of relatively lower pressure causing gradual load transfer from the liquid to the solid phase.

The stress on the capillary water disappears within a few days by being transferred to the surrounding gel. Similarly, the stress on the gel water disappears after a number of weeks. It appears that the pressure on the inter- and intracrystalline water acts almost indefinitely (Neville et al., 1983).

Furthermore, the reversible creep which occurs at an initially high rate and generally stabilises within a two month period after loading is governed by the migration of capillary and gel water. The irreversible creep takes place in the interparticle and inter- and intracrystalline spaces and continues for many years after the reversible creep has ceased (Neville et al., 1983).

2.4.4 Feldman and Sereda's hypothesis

This hypothesis suggests that the relocation of inter- and intralayer water is of utmost importance and the role of adsorbed water is not significant in the creep process. The application of a load to the concrete causes the removal of the inter- and intralayer water which results in a reduction in the layer thickness and spacing. This process is responsible for the reversible creep component (Feldman and Sereda, 1968 & 1969).

The irreversible creep occurs as a result of viscous flow of gel layers relative to each other, introducing a process of breaking and remaking of mechanical and chemical interparticle bonds (Feldman and Sereda, 1968 & 1969).

2.4.5 Kesler's hypothesis

Vaishnav and Kesler (1961) suggested that the mechanisms of seepage, delayed elasticity and viscous flow are responsible for creep at lower stresses.

The initially high creep rate is probably due to seepage which is reversible, provided the desorbed water is available for re-sorption. However, creep resulting from permanent changes in the arrangement of the gel particles due to the formation of new bonds and new gel particles by hydration during the intervening period is not reversible. In addition to the seepage, after the initial period, viscous deformation occurs at the points of contact of gel particles covered with adsorbed water. This displacement of the gel particles is irreversible. The delayed elastic behaviour may arise from the molecular diffusion of the amorphous components of the gel, acting in parallel with the more or less crystalline components (Neville et al., 1983).

2.5 The Influence of Aggregate Stiffness on Creep

In general, concrete is viewed as a two phase material consisting of the hardened cement paste and the aggregate. The role of the paste is to provide strength to the concrete and the aggregate is relied upon to provided bulk, rigidity and dimensional stability and normally reduce the cost. When concrete is subjected to an external stress, the resulting creep strain of the concrete is assumed to occur mainly in the paste if the aggregate does not undergo any time-dependent deformation of its own. The aggregate which is interspersed in the paste reduces creep deformation of the paste by diluting the paste and by restraining its creep.

As creep continues to occur in the paste, more of the applied load is progressively applied to the aggregate until a stage is reached where further deformation is governed by the aggregate stiffness. Results of recent research indicate that creep may also be affected by the lower density porous layer comprising the interfacial zone around the particles (Mindess and Alexander 1995) and (Alexander and Milne, 1995).

The following discussion describes how the creep of concrete is influenced by the aggregate type and, in particular, aggregate stiffness.

2.5.1 Aggregates outside South Africa

Troxell et al., (1958) studied the influence of aggregate type on creep. The results of their research, which clearly confirm the significant influence of aggregate type on creep strain, are shown in Figure 2.10.

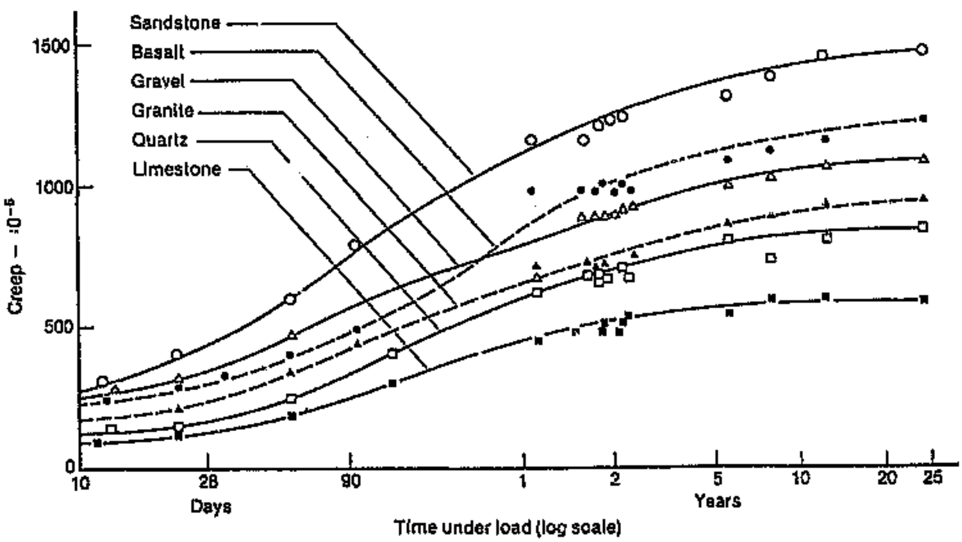


Figure 2.10 Creep of concretes made with different aggregates; a/c ratio = 5.67; w/c = 0.59; applied stress = 5.5 MPa (after Troxell et al., 1958)

It is evident from Figure 2.10 that, all intrinsic and extrinsic parameters being constant, the creep of sandstone concretes can exceed the creep of limestone concretes by a factor of up to 2.5. Research by Rusch et al, (1962), also showed that sandstone concretes exhibited a higher creep strain than concretes made with other aggregates. However, the order of increasing creep with the use of the different aggregates differed from that shown in Figure 2.10. This difference is probably attributable to the variance in the mineralogical and petrological composition (Alexander, 1994a) and physical properties of the aggregates used in the two projects.

The results of research by The Concrete Society (1974) in London clearly indicate that the higher the elastic modulus of an aggregate, the greater the restraint offered by the aggregate to the creep of the paste. This is shown in Figure 2.11 where it can be seen that the magnitude of creep of concretes containing low-modulus aggregates may be up to four times that of concretes with relatively stiffer aggregates. Furthermore, creep becomes insensitive to aggregate type in the case of aggregates with a modulus of elasticity in excess of 70 GPa (Alexander, 1994a).

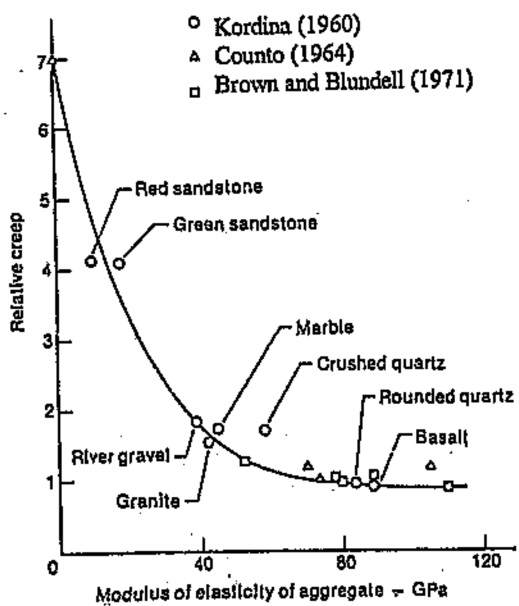


Figure 2.11 The effect of aggregate stiffness on creep of concrete (after The Concrete Society, 1974)

Kordina (1960) showed that a relationship exists between the absorption and elastic modulus of an aggregate. Since the elastic modulus of a material depends not only on the deformability of its constituents but also on the structure of the material, the absorption in this case may have effectively been the porosity of the material (Neville et al., 1983). Nevertheless, this relationship is shown in Figure 2.12.

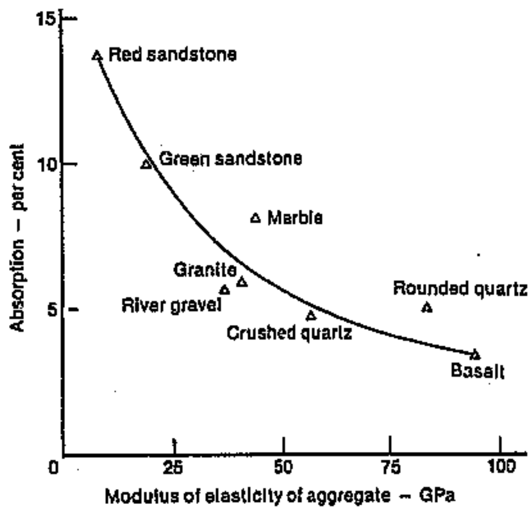


Figure 2.12 Relationship between absorption and modulus of elasticity of different aggregates (after Kordina, 1960)

Work by Soroka and Jaegermann (1972) on creep of low density aggregate concretes indicated an increase in creep with a decreasing aggregate modulus of elasticity. A comparison of low-density aggregate concrete and normal-density aggregate concrete on the basis of an equal stress : strength ratio showed the creep properties of the two concrete types to be essentially the same.

2.5.2 South African aggregates

Extensive research on the creep properties of eight most commonly used South African aggregate types from 23 sources was carried out by Davis and Alexander (1992).

The results of this investigation led to the establishment of relative creep values for the different aggregates, which are shown in Figure 2.13. It is evident from this figure that concretes with aggregates of the same generic origin from different sources exhibit different creep magnitudes.

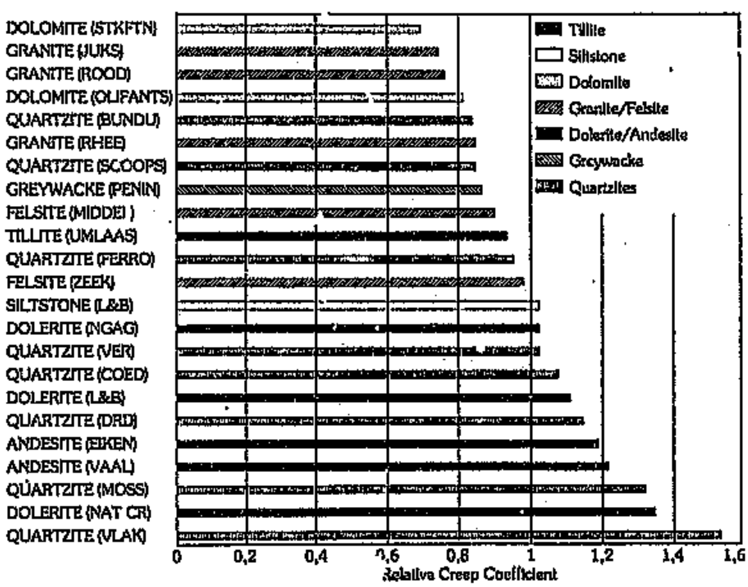


Figure 2.13 Relative creep of different South African aggregates (after Davis and Alexander, 1992)

Alexander (1993a) found no correlation between the magnitude of the creep of concrete and the elastic modulus of the aggregate used in the concrete.

2.6 Conclusions

There are many intrinsic and extrinsic factors that affect the creep of concrete, including aggregate stiffness. Research into the affect of these different factors on creep has led to the establishment of general relationships between creep magnitude and each of these factors. Such results have been used to the development of a number of models to explain the mechanisms of creep.

Since not one of the mechanism theories developed accounts for the observed facts, a number of hypotheses have been proposed by different researchers, on the basis of combining more than one mechanism, in an attempt to explain the phenomenon of creep.

To date no universally accepted mechanism or hypothesis has been established, probably as their is little evidence at a microstructural level to separate one mechanism from another (Yunping and Jennings, 1992).

The results of creep tests conducted on concretes comprising South African aggregate and binder types and exposed to a South African environment are discussed in Chapter 5 of this report. In this discussion particular attention is given to the influence of aggregate stiffness on the creep behaviour of these concretes.

CHAPTER 3

REVIEW OF CREEP PREDICTION METHODS

3.1 Levels of Creep Estimation

The magnitude of creep, which is required for design purposes, can be estimated at various levels. The choice of level depends on the type of structure and the time of prediction with regards to the information available. A three level approach proposed by Illston et. al., (1979) is briefly described below :

3.1.1 Lowest level

At this level rough estimates which are suitable only for approximate calculations are required. These estimates are based on a few input parameters, which are available at the design stage, such as characteristic compressive strength of concrete, member thickness and relative humidity. Such estimates are acceptable for structures that are not sensitive to deformations but are not in accordance with the degree of accuracy required for deformation-sensitive structures and hence would have to be revised at the intermediate or highest level when more detailed information is made available.

The level at which the final estimates are required is dependent on the nature of the structure.

3.1.2 Intermediate level

At this level estimates are made by utilising relatively complex creep prediction models which require input data which is available at the early stages of construction. This data typically includes information on the mix design such as 28 day compressive strength of the concrete, w/c ratio, binder type and age of loading.

3.1.3 Highest level

This level is relevant to structures where time dependent movements are critical. The data required is usually produced by means of comprehensive laboratory testing and mathematical and computer analyses. In the case of prestressed structures post-construction *in-situ* measurements can be used as a basis for adjusting the stresses in the tendons, thereby ensuring that the structure conforms with the design.

This chapter focuses on reviewing some of the better known methods which can be applied to predict creep strains, generally at the *intermediate level*, without the need for creep tests.

3.2 Creep Prediction Methods

3.2.1 General structure of methods

A total of seven different creep prediction methods are included in this investigation and reviewed below. These are the:

- British Standards Institution - Structural Use of Concrete, BS 8110 - Part 2 - (1985)
- American Concrete Institute (ACI) Committee 209 (1992)
- Standards Association of Australia - Australian Standard for Concrete Structures - AS 3600 (1988)
- Comité Euro-International Du Béton - Federation Internationale De La Precontrainte (CEB-FIP) Model Code (1970)
- CEB-FIP Model Code (1978)
- CEB-FIP Model Code (1990)
- International Union of Testing and Research Laboratories for Materials and Structures (RILEM) Model B3 (1995)

With the exception of the RILEM Model B3 (1995), the above models derive from structural design codes of practice and express creep strain as the product of the elastic deformation of the concrete (at the time of loading) and the creep coefficient.

The creep coefficient accounts for the effect of one or more intrinsic and/or extrinsic variables. The RILEM Model B3 (1995) is, by relative comparison, more complex than the design code models and has a different structure as it enables the calculation of separate compliance functions for the basic creep and drying creep (in excess of the basic creep). All the methods employ one or more nomograms and/or algebraic expressions to determine the creep strain. Table 3.1 shows which factors are accounted for in each of the prediction models.

3.2.2 The BS 8110 (1985) method

The British Standard method (1985) is contained in BS 8110. This method was earlier proposed by the British Concrete Society (1978) and is based on the CEB-FIP (1970) recommendations and has been incorporated into the SABS 0100 (1992) code. This method enables the estimation of final (30 year) creep strain (ε_{cc}) using the following equation:

$$\varepsilon_{cc} = \frac{\sigma}{E_c(t)}\phi^* \tag{3.1}$$

where,

σ	=	applied constant stress (MPa)
ϕ^*	=	final creep coefficient
$E_c(t)$	=	elastic modulus of the concrete at the time of loading (MPa)

Table 3.1 Summary of factors accounted for by different prediction methods

METHOD	Intrinsic Factors										Extrinsic Factors													
	Aggregate Type	A/C Ratio	Air Content	Cement Content	Cement Type	Concrete Density	Fine/Total Aggregate Ratio (Mass)	Slump	W/C Ratio	Water Content	Age at First Loading	Age of Sample	Applied Stress	Characteristic Strength at Loading	Cross-section Shape	Curing Conditions	Compressive Strength at 28 Days	Duration of Load	Effective Thickness	Elastic Modulus at Age of Loading	Elastic Modulus at 28 Days	Relative Humidity	Temperature	Time Drying Commences
BS 8110 (1985)	x										x		x	x					x	x		x		
ACI 209 (1992)			x			x	x	x			x		x				x	x	x		x	x		
AS 3600 (1988)						x					x		x				x	x	x		x	x		
CEB - FIP (1970)				x	x				x		x		x				x	x	x		x	x		
CEB - FIP (1978)					x						x		x				x	x	x		x	x		
CEB - FIP (1990)					x						x		x				x	x	x		x	x	x	
RILEM MODEL B3 (1995)		x		x	x				x	x	x	x	x		x	x	x	x	x	x	x	x	x	x

Determination of creep coefficients

The final creep coefficient (ϕ^*) is determined from Figure 3.1 (based on the CEB-FIP, 1970 recommendations) which account for the ambient relative humidity, age at loading and the effective thickness of the member under consideration.

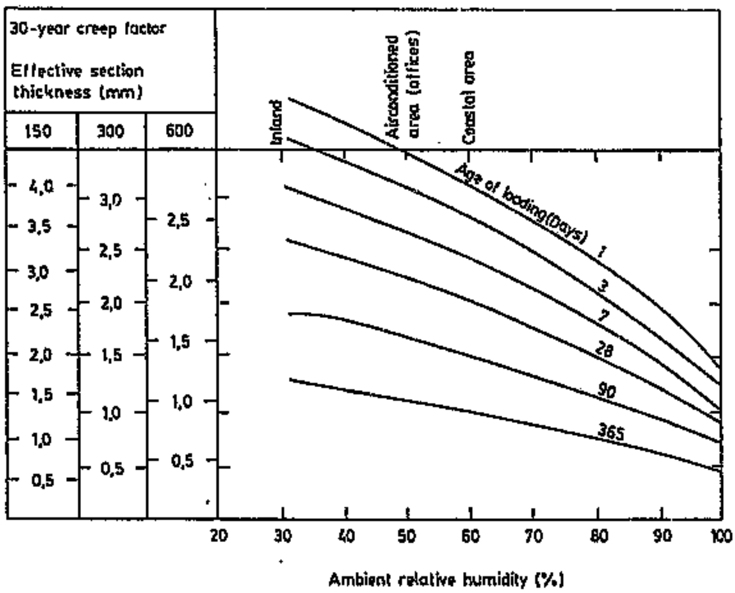


Figure 3.1 Effects of relative humidity, age of loading and section thickness upon creep factor (after BS 8110, 1985)

The effective thickness (in mm) is defined as twice the volume of the member divided by the exposed surface area. For a slab drying from both faces this reduces to slab thickness. In the case of members which are fully sealed or immersed in water, where only basic creep occurs, an effective section thickness of 600 mm should be used.

Determination of elastic modulus

Acco·ding to the BS 8110 (1985) method, the elastic modulus at the time of loading $E_c(t)$ is derived from the following two empirical expressions :

$$E_c(t) = E_{c,28}\left[0{,}4 + 0{,}6\frac{f_{cu}(t)}{f_{cu}(28)}\right] \tag{3.2}$$

$$E_{c,28} = K_0 + 0{,}2f_{cu}(28) \tag{3.3}$$

where,

$E_{c,28}$	=	static elastic modulus of elasticity at 28 days for normal weight concrete. This value is adjusted for lightweight concrete
$f_{cu}(t)$	=	compressive strength of the concrete at time t
$f_{cu}(28)$	=	28 day cube strength of the concrete in MPa
K_0	=	constant dependent on the stiffness of the aggregate and may be taken as 20 GPa for normal weight concrete.

Modifications by Davis and Alexander

Research conducted by Davis and Alexander (1992), on the influence of South African aggregate types on creep has led to a refinement of the BS 8110 (1985) method for certain aggregate types. These modifications entail:

- the application of a Relative Creep Coefficient for certain aggregate types, and

- the establishment of an expression for the determination of elastic modulus, which takes the aggregate type into account.

The Relative Creep Coefficient, obtained from Figure 2.13, is multiplied by the creep coefficient obtained from Figure 3.1 to obtain the final creep coefficient. This modification allows for the effect of the stiffness of the particular aggregate type on the final creep coefficient and is justified by the fact that the BS 8110 (1985) method is a simple predictive method and omits to account for a number of secondary variables such as temperature, curing conditions and w/c ratio (Davis and Alexander, 1992).

Tests conducted by Davis and Alexander (1992) to determine the elastic modulus (E) of concrete made with different South African aggregates have led to the establishment of the following expression for estimating E.

$$E = K_0 + \alpha f_{cu} \qquad (3.4)$$

where,

E	=	static modulus of elasticity for the particular age of concrete being considered in GPa
f_{cu}	=	the cube strength in MPa (generally the characteristic strength)
K_0	=	constant related to the stiffness of the aggregate in GPa

α = strength factor which is also related to the aggregate characteristics, expressed in GPa/MPa.

The values of K_0 and α have been determined for South African aggregates for different ranges of concrete age, namely 3 to 28 days and 6 months or older, as shown in Tables A.1 and A.2.

Rate of creep strain development

The BS 8110 (1985) method states that 40%, 60% and 80% of the final creep may be assumed to develop during the first month, six months and 30 months of loading, respectively. This, however, is only applicable when concrete is exposed to constant conditions of relative humidity (Davis and Alexander, 1992).

3.2.3 The ACI 209 (1992) method

The ACI Committee 209 (1992) method of creep prediction is identical to the ACI 209 (1978) method. According to this method, the creep strain at time t for a constant stress σ_0 first applied at age τ is :

$$\varepsilon_c(t) = \frac{\sigma_o}{E_c}\phi \tag{3.5}$$

where, ϕ is a creep coefficient and E_c is the elastic modulus at the time of loading, which is obtained from the following expression developed by Pauw (1990):

$$E_c(\tau) = \rho^{1.5}\, 0.043\, [f_c(\tau)]^{1/2} \tag{3.6}$$

where,

ρ	=	the density of the concrete (kg/m^3)
$f_c(\tau)$	=	the mean compressive strength at the time of loading (MPa) based on the average uniaxial compressive strength measured on cylinders.

The values of $E_c(\tau)$ given by this equation are applicable for applied stresses up to 40 per cent of the mean compressive strength of the concrete at the time of loading. This method also includes an equation for the prediction of concrete compressive strength at the time of loading which takes into account the cement type used and the method of curing employed.

Calculation of the creep coefficient

The following hyperbolic function represents the creep-time relationship :

$$\phi(t,\tau) = \frac{(t-\tau)^{0.6}}{10+(t-\tau)^{0.6}}\phi^*(\tau) \tag{3.7}$$

where, τ is the age of the concrete at first loading (in days); t is the age of the concrete (in days); $\phi^*(\tau)$ is the final creep coefficient and is expressed as :

$$\phi^*(\tau) = 2.35\gamma_1\gamma_2\gamma_3\gamma_4\gamma_5\gamma_6 \tag{3.8}$$

γ_1 to γ_6, are empirical correction factors (or partial coefficients), which account for the majority of parameters which are likely to influence the magnitude of creep. These factors include the age of concrete at the time of loading and the method of curing (moisture or steam), variations in relative humidity (γ_2), size and shape of the member (γ_3), slump of the fresh concrete (γ_4), ratio of fine aggregate to total aggregate by mass (γ_5) and air content in the concrete (γ_6). The equations used to calculate γ_1 to γ_6 are given below.

$$\gamma_1 = 1.25\tau^{-0.118} \quad \text{for } \tau > 7 \text{ days - for moist cured concrete} \tag{3.9}$$

or

$$\gamma_1 = 1.13\tau^{-0.094} \quad \text{for } \tau > 3 \text{ days - for steam cured concrete} \tag{3.10}$$

$$\gamma_2 = 1.27 - 0.0067\lambda \ \text{ for } \lambda > 40 \quad \text{where} \quad \lambda = \text{relative humidity (\%)} \tag{3.11}$$

Calculation of γ_3

$$h_o = 4V/S \tag{3.12}$$

where,

h_o = average thickness (mm)

V = volume (mm^3)

S = surface area (mm^2)

when $h_o \leq 150$ mm, γ_3 is obtained from Table 3.2.

Table 3.2 γ_3 correction factors applicable when $h_o \leq$ 150 mm (after ACI 209, 1992)

h_o(mm)	50	75	100	125	150
γ_3	1.3	1.17	1.11	1.04	1.00

When 150 mm $< h_o <$ 380 mm

$$\gamma_3 = 1.14 - 0.00092h_o \qquad \text{when } t - \tau \leq 365 \text{ days} \tag{3.13}$$

or

$$\gamma_3 = 1.10 - 0.00067h_o \qquad \text{when } t - \tau > 365 \text{ days} \tag{3.14}$$

and when $h_o \geq 380$ mm

$$\gamma_3 = 2/3\ \{1 + 1.13e^{-0.0213\ V/S}\} \tag{3.15}$$

<u>Calculation of γ_4 to γ_6</u>

$$\gamma_4 = 0.82 + 0.00264s \tag{3.16}$$

$$\gamma_5 = 0.88 + 0.0024\psi \tag{3.17}$$

$$\gamma_6 = 0.46 + 0.09a \quad \text{(but not less than 1.0)} \tag{3.18}$$

where,

s	=	slump of the fresh concrete (mm)
ψ	=	ratio of the fine aggregate to total aggregate by mass (%)
a	=	air content (%)

3.2.4 The AS 3600 (1988) method

This method is intended to apply to structures with a characteristic compressive strength at 28 days within the range of 20 to 50 MPa.

The general expression for predicting creep strain at any time $\varepsilon_c(t)$ due to a sustained stress σ_o (in MPa) first applied at age τ (in days) is given by :

$$\varepsilon_c(t) = \frac{\sigma_o}{E_c(\tau)}\phi \tag{3.19}$$

where, ϕ is a creep coefficient and $E_c(\tau)$ is the elastic modulus at the time of loading, which is obtained from the expression developed by Pauw (1990) which is given below. This is the same equation that is used in the ACI 209 (1992) method for estimating the value of E.

$$E_c(\tau) = \rho^{1.5}\, 0.043\, [f_c(\tau)]^{1/2} \tag{3.20}$$

where,

ρ	=	the density of the concrete (kg/m^3)
$f_c(\tau)$	=	the mean compressive strength at the time of loading (MPa) measured on cylinders.

The Australian Standard AS 3600 (1988) proposes the following mathematical expression for predicting the creep coefficient (ϕ) at time t as a result of a sustained stress first applied at age τ :

$$\phi(t, \tau) = k_2 k_3\, \phi_{cc.b} \tag{3.21}$$

where,

$\phi_{cc.b}$ = basic creep factor which is defined as the ratio of the ultimate creep strain to elastic strain for a specimen loaded at 28 days under a constant stress equal to 40 per cent of the compressive strength (f_c) of the concrete. This factor is obtained from the Table 3.3 below.

k_2 = creep factor coefficient (obtained from Figure 3.2) which depends on the relative humidity, time after loading and the hypothetical thickness (t_h). The hypothetical thickness (in mm) is equal to twice the cross-sectional area of the member (in mm^2) divided by the perimeter of the cross section exposed to drying (in mm).

k_3 = maturity coefficient (obtained from Figure 3.3) which depends on the age of the concrete at the time of loading and is obtained from the strength ratio $f_{cm}/f_c(28)$, where f_{cm} and $f_c(28)$ are the compressive strength at the time of loading and the compressive strength at 28 days, respectively. These compressive strengths are taken as the cylinder strengths.

Table 3.3 Basic creep factors (after AS 3600, 1988)

f_c (MPa)	20	25	32	40	50
$\phi_{cc.b}$	5.2	4.2	3.4	2.5	2.0

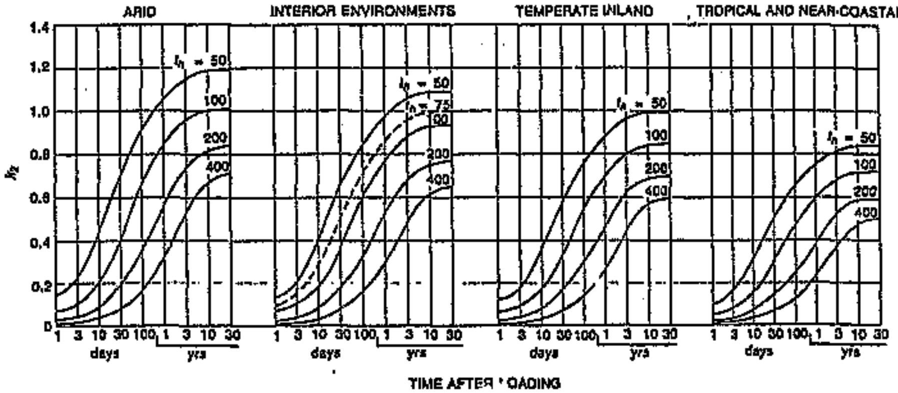


Figure 3.2 Creep factor coefficient (k_2) for various environments (after AS 3600, 1988)

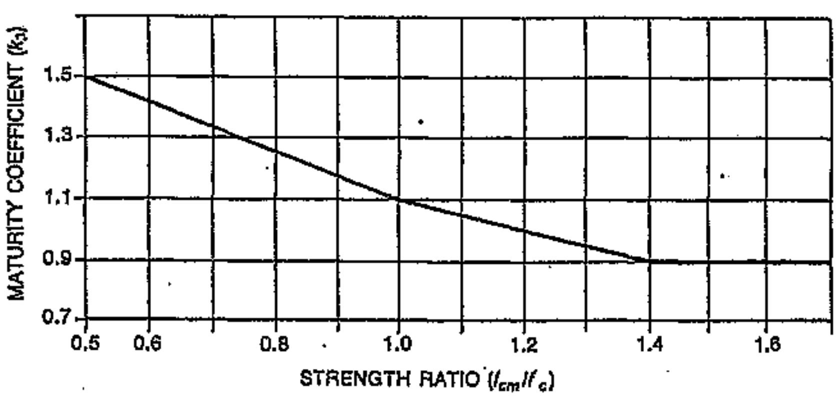


Figure 3.3 Maturity coefficient (k_3) values (after AS 3600, 1988)

3.2.5 The CEB-FIP Model Code (1970)

This method was proposed in 1970 but has been superseded by the CEB-FIP (1978) and CEB-FIP (1990) methods. According to Gilbert (1988), this model is still of importance as it is included in many Australian building codes and has been shown to be reasonably accurate (Hilsdorf and Muller, 1979).

The creep strain (ε_c) at time t, as a result of a constant sustained stress, σ_0 applied at time τ is predicted by applying the following equation :

$$\varepsilon_c(t,\tau) = \frac{\sigma_0}{E_{c28}}\phi(t,\tau) \tag{3.22}$$

Where E_{c28} is the longitudinal modulus of deformation at 28 days (taken as the mean value of the secant modulus) and is calculated, in MPa, using the following equation:

$$E_{c28} = 5940\,[f_{cu(28)}]^{1/2} \tag{3.23}$$

Where $f_{cu(28)}$ is the average compressive strength of the concrete at 28 days in MPa, measured on cylinders.

The creep coefficient to be used in equation 3.22 is calculated using the following expression :

$$\phi(t,\tau) = k_1 k_2 k_3 k_4 k_5 \tag{3.24}$$

The five coefficients (k_1 to k_5) are calculated by applying the following equations:

$$k_1 = 3 + 0.01\lambda - 0.0003\lambda^2 \tag{3.25}$$

$$k_2 = 0.45 + 1.76e^{-0.267\tau\text{^}0.44} \tag{3.26}$$

$$k_3 = [1.3 + 0.007c](w/c) - 0.85 \tag{3.27}$$

$$k_4 = 0.7 + 0.77e^{-0.009b} \qquad \text{where } b = 2(A_g/u) \tag{3.28}$$

$$k_5 = t^{0.8} / (t^{0.8} + 0.25b) \tag{3.29}$$

where,

λ	=	relative humidity (%)
τ	=	age of loading (days)
c	=	cement content (kg/m^3)
w/c	=	water/cement ratio
b	=	theoretical thickness (mm)
A_g	=	cross sectional area of member (mm^2)
u	=	perimeter of the cross section exposed to drying (mm)
t	=	time under load (days)

The validity of this method is restricted to normal quality concretes made with portland cement which are subjected to a maximum working stress equal to 40 per cent of their rupture stress as determined on cylinders.

3.2.6 The CEB-FIP Model Code (1978)

This method which is documented in the CEB-FIP Model Code (1978) and concisely summarised in Gilbert (1988), prescribes the same equation specified in the CEB-FIP (1970) for the prediction of creep strain. This equation predicts the creep strain (ε_c) at time t, as a result of a constant sustained stress, σ_0 applied at time τ, as follows.

$$\varepsilon_c(t,\tau) = \frac{\sigma_0}{E_{c28}}\phi(t,\tau) \tag{3.30}$$

Where E_{c28} is the longitudinal modulus of deformation at 28 days (taken as the mean value of the secant modulus) and is calculated, in MPa, using the following equation:

$$E_{c28} = 9500\ [f_{cm}]^{1/3} \tag{3.31}$$

Where f_{cm} is the average compressive strength of the concrete at 28 days in MPa, measured on cylinders.

However, one of the differences between this method and the CEB-FIP (1970) method is that this method enables the separate calculation of the reversible delayed elastic strain and irreversible flow components of the creep coefficient at any age t (including the final creep coefficient). The flow component is sub-divided into a component representing flow during the first 24 hours under load (rapid initial flow) and a subsequent flow component.

The creep coefficient, $\phi(t,\tau)$, is estimated from the sum of the delayed elastic strain, rapid initial flow and delayed flow components using the following equation.

$$\phi(t,\tau) = \phi_d\, \beta_d(t-\tau) + \beta_a(\tau) + \phi_f[\beta_f(t) - \beta_f(\tau)] \tag{3.32}$$

where,

t	=	age of specimen (in days) at which creep strain is required.
τ	=	age of concrete at first loading (in days)
ϕ_d	=	the final delayed elastic creep coefficient which is taken as the ratio of the final delayed elastic strain and the instantaneous strain at 28 days which is equal to 0.4
$\beta_d(t-\tau)$	=	describes the development of delayed elastic strain with time
$\beta_a(\tau)$	=	rapid initial flow
ϕ_f	=	flow coefficient

$\beta_f(t)$	=	function describing the development of delayed flow with time
$\beta_f(\tau)$	=	function to account for the age at application of load (when $t = \tau$).

The equations used to calculate the creep components are as follows :

$$\beta_d(t-\tau) = 0.73\left(1-e^{-0.01(t-\tau)}\right)+0.27 \tag{3.33}$$

$$\beta_a(\tau) = 0.8\left[1-\frac{\tau^{0.73}}{5.27+\tau^{0.73}}\right] \tag{3.34}$$

$$\phi_f = \phi_{f1} + \phi_{f2} \tag{3.35}$$

where,

$$\phi_{f1} = \frac{1}{9}\left[0.0002h^3 - 0.043h^2 + 2.57h\right] - 2.2 \tag{3.36}$$

$$\phi_{f2} = 1.12\left[1+e^{-0.044h_o^{0.58}}\right] \tag{3.37}$$

$$\text{where } h_o = \lambda\frac{2A_c}{u} \tag{3.38}$$

$$\text{and } \lambda = 1+0.00049e^{0.1h} \text{ when } h \leq 98 \tag{3.39}$$

$$\lambda = 30 \qquad \text{when } h = 100 \tag{3.40}$$

$$\beta_f(t)-\beta_f(\tau) = \left(\frac{t}{t+H_f}\right)^{1/3} - \left(\frac{\tau}{\tau+H_f}\right)^{1/3} \quad \text{- for all values of } h_o \tag{3.41}$$

Alternatively, equation 3.42 by Gilbert (1988) may be used for more accurate results in cases where 50 mm $\leq h_o \leq$ 1600 mm

$$\beta_f(t)-\beta_f(\tau) = \left[\frac{t^\alpha}{t^\alpha+\beta}\right]^{1/3} \tag{3.42}$$

where,

$$\alpha = 0.8+0.55e^{-0.003ho} \quad (3.43)$$

$$\beta = 770+210e^{-0.0043ho} \quad (3.44)$$

τ = age of concrete at first loading (days)

t = age of specimen (days)

h = relative humidity (%)

h_o = notional thickness (mm)

λ = humidity coefficient

A_c = cross-sectional area of member (mm^2)

u = perimeter exposed to drying (mm)

H_f = constant depending on the notional thickness give in Table 3.4

Table 3.4 Values of H_f for various notional thicknesses (after CEB-FIP, 1978)

h_o (mm)	50	100	200	400	800	1600
H_f (days)	330	425	570	870	1500	2500

In cases where the ambient temperature during curing is significantly different from 20 °C, the age of the concrete (t) to be used in equations 3.33 and 3.41 or 3.42 is adjusted to an effective age (t_e). This adjustment is dependent on the mean daily temperature of the concrete and the type of cement used, as detailed below.

$$t_e = \frac{\alpha}{30}\sum_{0}^{t_m}\left[(T(t_m)+10)\Delta t_m\right] \tag{3.45}$$

where,

T	=	mean daily temperature of the concrete (°C) during a period Δ_{tm} (days)
α	=	1 for normal and slow hardening cements
	=	2 for rapid hardening cements
	=	3 for rapid hardening high strength cements

This method of creep prediction is only applicable to concrete subjected to a compressive stress not exceeding 40 percent of the compressive strength. Furthermore, the numerical values predicted should be considered as representative values liable to vary by 20 per cent in either direction.

3.2.7 The CEB-FIP Model Code (1990)

This method, which is described in the CEB-FIP Model Code 1990 (First Draft), may be applied to predict creep strain after a given duration of loading (ε_{cc} (t,t_o)) from the following equation:

$$\varepsilon_{cc}(t,t_o) = \frac{\sigma_c(t_o)}{E_c}\phi(t,t_o) \tag{3.46}$$

where,

σ_o = the constant stress sustained at time t_o

$\phi(t,t_o)$ = creep coefficient

E_c = elastic modulus at 28 days

The elastic modulus at 28 days (E_c) is calculated using an empirical equation which takes into account the characteristic strength of the concrete at 28 days in MPa (f_{cm}) as follows.

$$E_c = 10^4[f_{cm}]^{1/3} \tag{3.47}$$

The characteristic strength (f_{cm}) is based on the uniaxial compressive strength of cylinders, which are 150 mm in diameter and 300 mm in height, stored in water at 20 ± 2 °C and tested at 28 days. The approximate cylinder strengths of various cube strengths are given in the Table 3.5.

Table 3.5 Characteristic cube and cylinder strength values (after CEB-FIP,1990)

f_{cm} - cube	15	25	37	50	60	70	85	95
f_{cm} - cylinder	12	20	30	40	50	60	70	80

The creep coefficient, $\phi(t,t_o)$ is estimated from :

$$\phi(t,t_o) = \phi_o \beta_c (t - t_o) \tag{3.48}$$

where,

ϕ_o	=	the notional creep coefficient
β_c	=	coefficient describing the development of creep with time
t	=	age of concrete (days) at the moment considered
t_o	=	age of concrete at loading (days)

The notional creep coefficient (ϕ_o) may be estimated using the following equations:

$$\phi_o = \phi_{RH}\, \beta(f_{cm}) \beta(t_o) \tag{3.49}$$

with :

$$\phi_{RH} = 1 + \frac{1 - \frac{RH}{100}}{0.1(h_o)^{1/3}} \quad \text{where} \quad h_o = \frac{2A_c}{u} \tag{3.50}$$

$$\beta(f_{cm}) = \frac{16.8}{\sqrt{f_{cm}}} \tag{3.51}$$

$$\beta(t_o) = \frac{1}{0.1 + t_o^{0.2}} \tag{3.52}$$

The development of creep with time, $\beta_c(t-t_o)$ is given by :

$$\beta_c(t-t_o)=\left[\frac{(t-t_o)}{\beta_H+(t-t_o)}\right]^{0.3} \tag{3.53}$$

with

$$\beta_H = 1.5\left[1+(0.012RH)^{18}\right]h_o + 250 \qquad \leq 1500\text{mm} \tag{3.54}$$

where,

RH	=	relative humidity of the ambient environment (%)
h_o	=	notional size of member (mm)
A_c	=	cross-sectional area of member (mm^2)
u	=	the perimeter of the member in contact with the atmosphere (mm)
f_{cm}	=	mean compressive strength of concrete at 28 days (MPa)
t_o	=	age of concrete at loading (days) (adjusted in case of $\beta(t_o)$ – see below)
t	=	age of concrete (days) at moment considered

Adjustments to the age of loading

The age of concrete at loading used to determine $\beta(t_o)$ (equation 3.52 above) is adjusted by applying equation 3.55 (below) which accounts for the degree of hydration resulting at the age of loading. The degree of hydration is affected by the cement type used in the concrete.

$$t_o = t_{o,T}\left[\frac{9}{2+t_{o,T^{1,2}}}+1\right]^{\alpha} \qquad t_o \geq 0.5 \tag{3.55}$$

where,

α	=	-1 for slow hardening cements, SL;
	=	0 for normal or rapid hardening cements, N; R;
	=	1 for rapid hardening high strength cements, RS;

$t_{o,T}$	=	age of concrete at loading (days) adjusted to account for elevated or reduced temperatures on the maturity of the concrete according to the equation 3.56.

$$t_T = \sum_{i=1}^{n} \exp\left[-\left(\frac{4000}{273+T(\Delta t_i)}-13.65\right)\cdot \Delta t_i\right] \tag{3.56}$$

where,

t_T	=	temperature adjusted concrete age which replaces t in the corresponding equations
$T(\Delta t_i)$	=	temperature (° C) during the time period Δt_i
Δt_i	=	number of days during which a temperature T prevails.

This method also makes an allowance for the non-linearity of creep at stress levels in the range of $0,4f_{cm}(t_o) < \sigma_c < 0,6f_{cm}(t_o)$ by using the following equations:

$$\phi_{o,k} = \phi_o e^{1.5(k-0.4)} \qquad \text{for } 0.4 < k \leq 0.6 \tag{3.57}$$

$$\phi_{o,k} = \phi_o \qquad \text{for } k \leq 0.4 \tag{3.58}$$

where,

$\phi_{o,k}$ = non-linear notional creep coefficient, which replaces ϕ_o in equation 3.48

k = stress-strength-ratio σ_c/f_{cm}

This method, which is simpler than those proposed in the CEB-FIP 1970 and 1978 codes, is valid for ordinary structural concrete with a compressive strength greater than 12 MPa but not exceeding 80 MPa and which is subjected to a maximum stress/strength ratio of 0.4 and exposed to mean relative humidities in the range 40 to 100 per cent at mean temperatures from 5 °C to 30 °C.

3.2.8 The RILEM Model B3 (1995)

The creep and shrinkage prediction model for analysis and design of concrete structures – model B3 (1995) has been published as a draft RILEM Recommendation. This model is based on the work of Bazant and Baweja (1994) and represents the third major update of the models previously developed by Bazant and Panula (1978 & 1979) and Bazant et al., (1991a & b and 1992a, b and c) at North-Western University. This model has been calibrated by a computerised data bank comprising data (about 15 000 points) obtained from different laboratories throughout the world and conforms with the recently formulated RILEM TC-107 (1995) guidelines.

The equations included in this method are based on imperial units. These equations have been converted to apply to SI unit inputs in the description of this method below.

The main notations used in the expressions constituting this model are defined as follows:

t	=	time, representing the age of concrete in days
τ	=	age at loading, in days
t_o	=	age when drying begins, in days (only $t_o \leq \tau$ is considered)

$C_0(t,\tau)$	=	compliance function for basic creep only in 10^{-6} MPa^{-1}
$C_d(t, \tau, t_o)$	=	compliance function for additional creep due to drying in 10^{-6} MPa^{-1}
ε_{sh}, $\varepsilon_{sh\infty}$	=	shrinkage strain and ultimate (final) shrinkage strain, always given in 10^{-6}; ε_{sh} considered negative (except for swelling, for which the sign is positive)
h	=	relative humidity of the environment (expressed as a decimal number, not as a percentage, $0 \leq h \leq 1$)
S(t)	=	time function for shrinkage
τ_{sh}	=	shrinkage half-time (in days)
D	=	2v/s = effective cross-section thickness in mm
v/s	=	volume-to-exposed surface ratio in mm
c	=	cement content of concrete in kg/m^3
w/c	=	ratio (by weight) of water to cementitious material
w	=	(w/c)c = water content of concrete in kg/m^3
a/c	=	ratio (by weight) of aggregate to cement
f'_c	=	28-day standard cylinder compression strength in MPa (if only design strength f_{ck} is known, then $f'_c = f_{ck} + 8{,}27$ MPa)
σ	=	stress applied in MPa
$E(t_o)$	=	elastic modulus of concrete at age when drying begins (t_o), in MPa

$E(t)$	=	elastic modulus of concrete at age t in MPa
$E(\tau)$	=	elastic modulus of concrete at time of loading in MPa

The application of this model enables the separate calculation of basic creep and drying creep.

Basic creep

The basic creep compliance (function) at time t due to a constant uniaxial stress σ applied at time τ is:

$$C_o(t,\tau) = 145.033\left[q_2\, Q(t,\tau) + q_3\, \ell n\left[1+(t-\tau)^n\right] + q_4 \ell n\left(\frac{t}{\tau}\right)\right] \tag{3.59}$$

where,

m	=	0.5	
n	=	0.1	
q_2	=	$1.278c^{0.5}(f_c)^{-0.9}$	(3.60)
q_3	=	$0.29\,(w/c)^4\, q_2$	(3.61)
q_4	=	$0.14(a/c)^{-0.7}$	(3.62)
$Q(t,\tau)$	=	binomial integral calculated from the following approximate equation which is derived by Bazant and Prasannan (1989).	

$$Q(t,\tau) = Q_f(\tau)\left[1+\left(\frac{Q_f(\tau)}{Z(t,\tau)}\right)^{r(\tau)}\right]^{-1/r(\tau)} \tag{3.63}$$

with

$$r(\tau) = 1.7\,(\tau)^{0.12} + 8 \tag{3.64}$$

$$Z(t,\tau) = (\tau)^{-m}\ln[1+(t-\tau)^n] \tag{3.65}$$

$$Q_f(\tau) = [0.086(\tau)^{2/9} + 1.21(\tau)^{4/9}]^{-1} \tag{3.66}$$

Drying creep

The compliance fu...tion for additional creep due to drying $C_d(t,\tau,t_o)$ is given below. This function is only applicable in situations where drying commences prior to or at the same time as the load application ($\tau \geq t_o$).

$$C_d(t,\tau,t_o) = 145.033q_5[e^{-8H(t)} - e^{-8H(\tau)}]^{1/2} \tag{3.67}$$

in which

$$H(t) = 1 - (1-h)S(t) \tag{3.68}$$

and

$$q_5 = 5219.52f_c^{\prime -1}\,\varepsilon_{sh\infty}^{-0.6} \tag{3.69}$$

where,

$$S(t) = \tanh\left[\frac{t - t_o}{\tau_{sh}}\right]^{1/2} \tag{3.70}$$

$$\tau_{sh} = 1.55x10^{-3}k_t(k_sD)^2 \tag{3.71}$$

D	=	2v/s = effective cross section thickness (in mm)
k_s	=	cross section shape factor which equals 1.00 for an infinite slab, 1.15 for an infinite cylinder, 1.25 for an infinite square prism, 1.30 for a sphere and 1.55 for a cube.

$$k_t = 54.98t_o^{-0.08}f'_c{}^{-1/4} \tag{3.72}$$

$$\varepsilon_{sh\infty} = \varepsilon_{s\infty}\frac{E(t_o)88034.81}{E(t)145.033(t_o + \tau_{sh})} \tag{3.73}$$

$$\varepsilon_{s\infty} = \alpha_1\alpha_2\{1.903x10^{-2}w^{2.1}f'_c{}^{-0.28} + 270\} \quad \text{(in } 10^{-6}\text{)} \tag{3.74}$$

α_1	=	1.0 for type I cement (ordinary portland cement)
	=	0.85 for type II cement
	=	1.1 for type III cement
α_2	=	0.75 for steam cured specimens
	=	1.0 for specimens cured in water or at 100% relative humidity
	=	1.2 for specimens sealed during curing

$$E(t) = E(28)\left(\frac{t}{4+0.85t}\right)^{1/2} \tag{3.75}$$

$$E(28) = 4733\sqrt{f'_c} \tag{3.76}$$

Calculation of creep strain

Drying plus basic creep strain $= [C_o(t,\tau)+C_d(t,\tau,t_o)]\sigma$ (3.77)

Basic creep strain $= [C_o(t,\tau)]\sigma$ (3.78)

Calculation of creep coefficient

The creep coefficients for basic plus drying creep ϕ_{bd} and for basic creep only ϕ_b are :

$$\phi_{bd} = [C_o(t,\tau) + C_d(t,\tau,t_o)]E(\tau) \tag{3.79}$$
$$\phi_b = [C_o(t,\tau)]E(\tau) \tag{3.80}$$

Applicability of model

This model is intended for portland cement concretes for the following parameter ranges:

17 MPa $\leq f_c \leq$ 70 MPa

160 kg/m^3 $\leq c \leq$ 720 kg/m^3

$0.3 \leq w/c \leq 0.85$

$2.5 \leq a/c \leq 13.5$

3.3 Conclusions

The seven creep prediction models reviewed in this chapter are all empirically based and vary with regards to their complexity and the number of factors that they account for. Of these methods, the BS 8110 (1985) method prescribes the simplest procedure and accounts for the least number of intrinsic and/or extrinsic variables. By contrast, the RILEM Model B3 (1995) employs the most complex procedure and considers the most variables. Furthermore, only the BS 8110 (1985) method directly accounts for the stiffness of the aggregate.

With the exception of the RILEM Model B3 (1995), all the methods reviewed in this chapter derive from structural design codes of practice and express creep strain as the product of the elastic deformation of the concrete (at the time of loading) and the creep coefficient. The structure of the RILEM Model B3 differs from that of the design code models as this model enables the calculation of separate compliance functions for the basic creep and drying creep (in excess of the basic creep), which may be added to obtain the total creep.

Chapter 6 assesses the accuracy of each of the seven models when applied to South African conditions, by comparing the values of the measured creep strains exhibited by the specimens in this investigation against those predicted at the corresponding ages by the different models. In this assessment, particular attention is given to the influence of the aggregate stiffness on the predicted creep values.

CHAPTER 4

EXPERIMENTAL DETAILS

4.1 Introduction

This chapter gives details on the materials and equipment used as well as the experimental techniques employed in this investigation. The material details include the cement and aggregate types and their sources as well as the material proportions comprising the different mixes. The experimental details include descriptions on the preparation of the concrete samples and tests conducted on the hardened concrete or on rock samples of the aggregates used in the concretes.

4.2 Materials

4.2.1 Cement

A single batch of CEM I 42,5 cement, complying with the SABS ENV 197-1 (1992) specification, was received from the Dudfield factory of Alpha Cement and used for all the tests carried out in this investigation.

4.2.2 Aggregates

Three aggregate types from different Hippo Quarries sources in South Africa were used in this investigation. These were quartzite from the Ferro quarry in Pretoria, granite from the Jukskei quarry in Midrand and andesite from the Eikenhof quarry in Johannesburg.

For each of the three aggregate types used, the coarse and fine aggregate were from the same source. The coarse aggregate was 19 mm nominal size and the fine aggregate was a crusher sand. The results of a grading analysis conducted on each of the crusher sands are given in Figures B.1 to B.3 in the appendix These crusher sands comply with the grading requirements for fine aggregate specified in SABS 1083 (1994), except in the case of the andesite where the dust content exceeds the permissible value (ten per cent) by approximately three per cent.

4.3 Laboratory Procedures

4.3.1 Mix proportions

The mixes were designed with the intention of investigating the following two aspects of creep behaviour:

- the difference in the creep behaviour of concrete with varying w/c ratios, and

- the differences in the creep behaviour of concretes with the same w/c ratio but different aggregate type.

Trial mixes were prepared to establish the water content which would yield a slump of approximately 50 mm when using a w/c ratio of 0.4 and quartzite aggregate. The quartzite aggregate was selected on the basis of its fineness modulus coincidentally being the average of the fineness modulus values of the three different crusher sands. The water content decided upon was 195 l/m^3.

Thereafter, a total of six mixes were designed in accordance with the C & CI method (formerly PCI method) which is described by Daly (1991). These comprised two mixes with w/c ratios of 0.56 and 0.4., for each of the three aggregate types included in the investigation. For each mix, a constant water content of 195 l/m^3 was used. The w/c ratios of 0.56 and 0.4 were chosen to respectively represent typical medium and high strength concretes used in practice. Table 4.1 shows the mix proportions of the six mixes and the slump values obtained for the concretes.

Table 4.1 Mix proportions and slump test results of the concrete used in this investigation

Aggregate Type	Quartzite		Granite		Andesite	
Mix Number	Q1	Q2	G1	G2	A1	A2
Water (l/m^3)	195	195	195	195	195	195
Cement (kg/m^3)	348	488	348	488	348	488
19 mm Stone (kg/m^3)	1015	1015	965	965	1135	1135
Crusher Sand (kg/m^3)	810	695	880	765	860	732
W/C Ratio	0.56	0.4	0.56	0.4	0.56	0.4
A/C Ratio	5.24	3.50	5.30	3.55	5.73	3.83
Slump (mm)	90	50	115	70	95	55

4.3.2 Mixing procedure

Sufficient concrete of each mix was prepared to fill three 101.6 x 101.6 x 700 mm beam moulds and six 101.6 mm cube moulds. A 100 kg mixer was charged with the materials in the order sand, cement, stone and water. The dry materials were mixed and then water was added over a period of approximately one minute. Thereafter, mixing was continued for a further three minutes. The slump tests were carried out immediately after mixing.

4.3.3 Compaction

All the moulds were loosely filled with concrete and were hand held on a mechanical vibrating table for approximately 10 seconds. Concrete was added to the moulds which were then held on the vibrating table for a further 30 to 60 seconds, depending on the consistency of the mix.

4.3.4 Curing

The concrete filled moulds were covered with plastic sheets and the temperature in the room was kept constant at 19 ± 1 °C for a period of approximately 20 hours. Thereafter, the concrete was removed from the moulds and placed in a water bath. The temperature of the water in the bath was maintained at 22 ± 1 °C.

Approximately 20 days after casting, each concrete beam was accurately cut to form three 101.6 x 101.6 x 200 mm prisms. The end faces of each prism were ground in a high speed facing machine to ensure that they were parallel to each other. Hence, nine prisms were obtained for each of the six mixes. The prisms were then placed back into the curing bath.

4.3.5 Preparation of specimens

At approximately 21 days after casting, the prisms were removed from the curing bath and Demec targets were glued onto two opposite formed sides of each prism, on a vertical axis symmetrically about the middle of the specimen, to accommodate a 100 mm Demec strain gauge. A quick-setting glue (Schnellklebstoff X 60 Epoxy Glue) which adheres to wet concrete was used for this purpose. After the glue had set (approximately 15 minutes after application) the prisms were returned to the curing bath where they remained for a total of 28 days after casting.

4.4 Testing of Hardened Concrete

4.4.1 Compressive strength

The six cubes cast for each mix were tested in uniaxial compression in an Amsler type 103 compression testing machine which has a capacity of 2 000 kN. Three cubes were tested at seven days and three at 28 days after casting.

The cubes were removed from the bath immediately before testing and weighed to the nearest 5 g, after the excess water had been wiped from the surface. The cubes were then placed in the testing apparatus. The load was applied at a rate of approximately 150 kN per minute. The failure load was recorded to the nearest 1 kN.

4.4.2 Creep and associated shrinkage tests

The prisms were removed from the curing bath at age of 28 days after casting. Of the nine prisms of each mix, three were left unsealed and subjected to a creep test (in loading frames), three were sealed and subjected to a creep test and the remaining three were left unsealed, unloaded and placed in the creep room to monitor shrinkage.

The sealed specimens were covered with a 5 mm (minimum) thick coat of water based bitumen and wrapped in aluminium foil. The trade name of the bitumen is ABE Super Laykold Rubberised Bitumen Waterproofer. All prisms were cured in water for 28 days and creep loads were applied immediately after this curing period.

Creep tests

Creep frames. These frames were developed by Ballim (1983) and are based on the ASTM C512-76 (1976) creep frame, except the load is applied by a hydraulic flat jack instead of a compressed spring. Figures 4.1 and 4.2 (overleaf) show details of the creep frames. Each frame was loaded by pumping oil into the flat jack by means of a hand pump. The load was maintained using a nitrogen pressure accumulator and monitored using a proving ring and dial gauge arrangement. The purpose of the dummy specimens at the top and bottom of the column was to eliminate the effects of end restraint from the loading plates on the specimens being tested. By means of an air conditioner and humidifier, the temperature and relat' /e humidity in the room in which the frames w're house was kept between 23 ± 3 °C and 65 ± 5 °C, respectively.

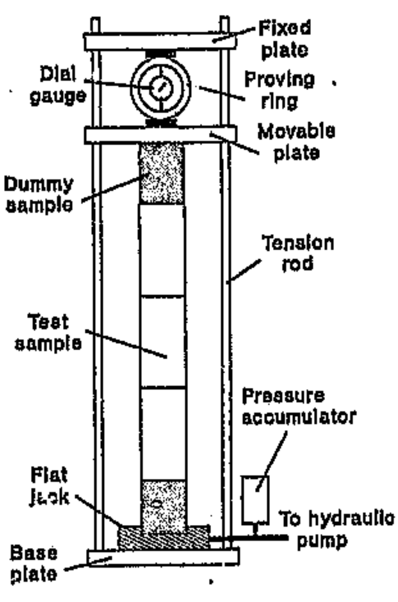


Figure 4.1 Schematic arrangement of the creep loading frame (after Ballim, 1983)

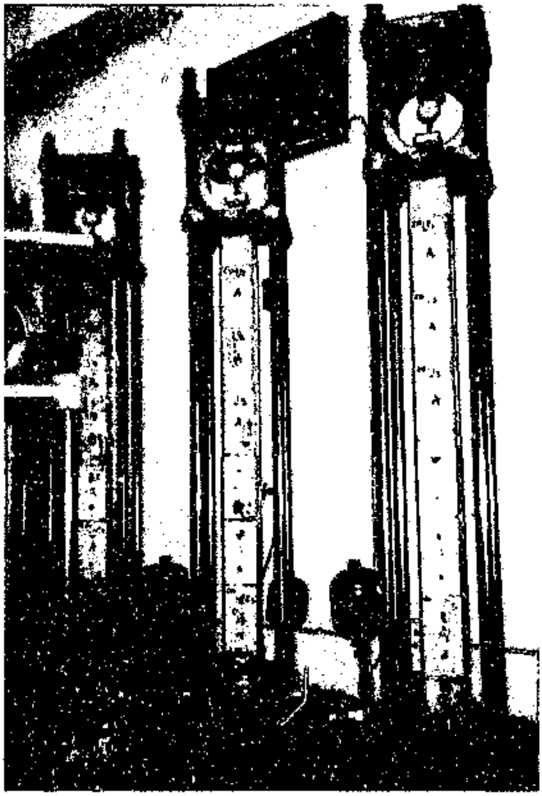


Figure 4.2 Loaded prisms in creep frames

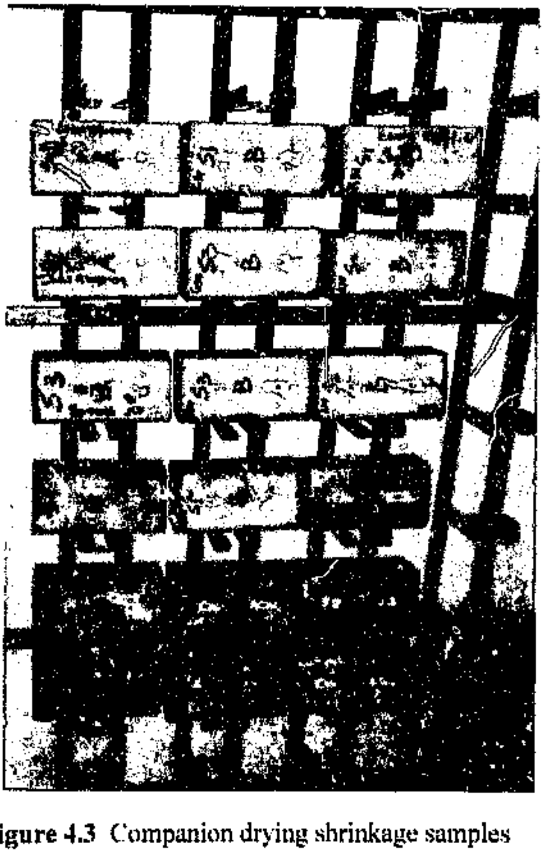


Figure 4.3 Companion drying shrinkage samples

The six creep specimens (three sealed and three unsealed) of each mix were placed in separate creep frames. The unsealed specimens were placed on top of the three sealed specimens. The joints at the ends of the sealed specimens which were in contact with other specimens were covered to prevent moisture loss, by means of an adhesive tape which was placed around the prism perimeter at the joint.

Loading. The 28 day compressive strength tests (cube tests) were conducted immediately before the frames were loaded. The prisms in each of the six creep frames were subjected to a constant stress equal to 25 per cent of the 28 day compressive strength of the relevant mix. Note that at higher stress levels of above 40 to 50 per cent of short term strength, the creep mechanisms are different as significant load - induced microcracking occurs (Alexander, 1994a). The stresses were maintained to an accuracy of ± 0,5 MPa for a period of six months. Specific details on the loading procedure are discussed in the section giving information on the measurements (below).

Shrinkage tests

The shrinkage samples (three for each mix) were placed on a rack in the same room as the creep samples and, in order to ensure a drying surface area equivalent to the creep samples, the two 100 mm square ends were dipped in warm wax to prevent drying from these surfaces. The rack, which is shown in Figure 4.3, is designed to facilitate air movement around all the specimens and a minimum distance of 10 mm between neighbouring specimens.

Measurements

After the specimens were stacked in the creep frame, readings were taken during the loading procedure described below:

- The load corresponding to a stress/strength ratio of 25 per cent (σ_a) was applied in the frame, maintained for 60 seconds, and then unloaded;
- Thirty seconds later, a pre-load of approximately 1 MPa (σ_b) was applied and maintained for 60 seconds;
- The load was increased to σ_a, maintained for 60 seconds and unloaded to σ_b;
- Thirty seconds later a set of readings was taken (at σ_b) and regarded as the zero-strain readings;
- The load was increased to σ_a and a set of readings was taken within ten minutes. These readings were taken as being the immediate elastic deflections (ASTM C512-76, 1976).

After the above loading procedure, the total strains were determined daily for one week, weekly until the end of one month, and approximately monthly thereafter. The shrinkage strain of the unsealed (companion) specimens was measured on the unloaded specimens which were exposed to the same environment as the loaded specimens. The zero-strain readings of the shrinkage specimens was taken immediately after the elastic strain readings of the creep specimens. Thereafter, readings on the shrinkage specimens were taken at the same time as those taken on the creep specimens.

At each measuring period, the strain of each prism was taken as the average of the strains measured on the two opposi e faces of the prism. The strain of each group of prisms, that is the three unsealed prisms or three sealed prisms or three companion shrinkage prisms of a particular mix, was taken as the average of the strains of the prisms in that group.

4.4.3 Determinatiou of elastic moduli of the aggregates

Measurements of aggregate stiffness were carried out on boulders which were collected from the area of each quarry where the aggregate, used in the concrete specimens in this investigation, derived from. Therefore, the stiffness of each rock type as determined on the boulders was taken to be representative of the stiffness of the relevant aggregates used in the concrete specimens.

Two rock boulders were collected from each of the quartzite (Ferro) and andesite (Eikenhof) quarries for the determination of the elastic modulus of the rock. Since the appearance of the rocks observed in the Granite (Jukskei) quarry appeared quite variable, two boulders of two different looking rocks occurring in that quarry were collected for testing.

Three cores measuring 42 mm in diameter and 82 mm long were cut from each set of two boulders and these were tested, according to the procedure described in BS 1881 (1983), to determine the elastic modulus of the aggregates used in this investigation. Two LVDT displacement gauges were attached diametrically opposite each other on each core and strain measurements were taken over a length of 50 mm. The cores were tested in the Amsler type 103 compression testing machine which has a capacity of 2 000 kN. The load and axial deformations of the specimens were autographically recorded by a Graphtech Data Recorder on an XY plotter over one cycle of loading and unloading. The cores were loaded to a maximum stress equal to approximately 25 per cent of the average unconfined compression strength values determined for the relevant aggregate types by Davis and Alexander (1992). Table 4.2 shows the average unconfined compressive strength values determined by Davis and Alexander (1992) and the maximum stresses exerted on the rock cores in this investigation.

Table 4.2 Average unconfined compressive strength values for the different aggregate types and maximum stresses applied to the rock samples

Aggregate Type	Unconfined Compressive Strength (UCS) (after Davis and Alexander, 1992)		Stresses Applied to Rock Samples		Applied Stress as a Percentage of Mean UCS
	Range (MPa)	Mean (MPa)	Core Number	Stress (MPa)	
Quartzite (Ferro)	105 - 394	250	1/1	61.5	24.6
			1/2	58.8	23.5
			1/3	61.4	24.6
Granite (Jukskei)	70 - 310	190	1/1	47.3	24.9
			1/2	48.6	25.6
			1/3	47.5	25.0
			2/1	47.4	24.9
			2/2	47.9	25.2
			2/3	46.9	24.7
Andesite (Eikenhof)	516 - 538	527	1/1	123.6	23.5
			1/2	129.4	24.6
			1/3	123.9	23.5

The results of all the tests conducted on the concrete specimens and on the rock samples are presented and discussed in the following chapter.

CHAPTER 5

RESULTS AND DISCUSSION

5.1 Compressive Strength of Concrete

The average seven and 28 day com strengths of the concrete cubes comprising the different aggregates used in the research are shown according to the two w/c ratios in Figure 5.1. The individual results of the cubes tested are given in Appendix C.

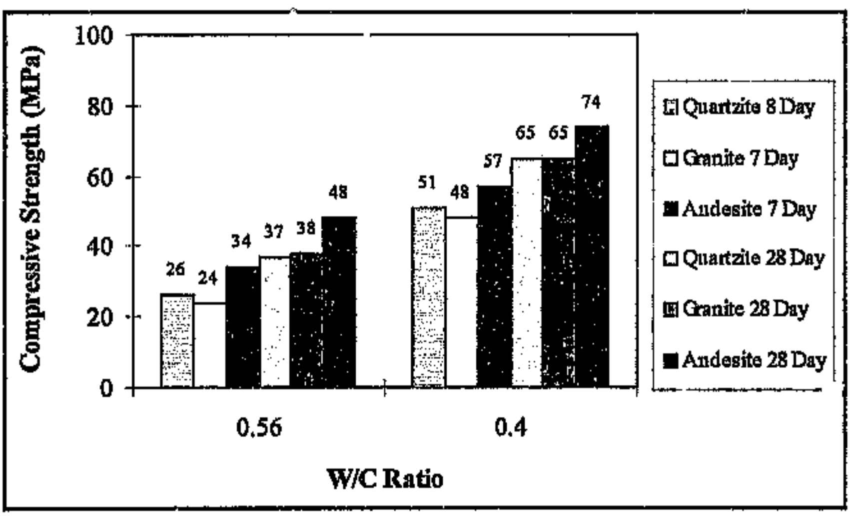


Figure 5.1 Average cube strengths of concretes with different aggregates for the two w/c ratios

It is clear from the figure above that the trend is for the andesite aggregate to yield relatively high strengths. Furthermore, the strengths of the quartzite and granite are generally of similar magnitude. This confirms that aggregates have a definite influence on the compressive strength of concrete. This was also noted by Hannant (1968), Ballim (1983) and Davis and Alexander (1992).

5.2 Measured and Inferred Strains

5.2.1 Measured strains

The strains measured on the laboratory specimens at the time periods mentioned in paragraph 4.4.2, for each of the six mixes (Q1, Q2, G1, G2, A1 and A2), together with the temperature and relative humidity values are presented in Tables D.1 to D.6 in the appendix. These strains comprise the shrinkage strains exhibited by the unloaded companion specimens and the total strains of the loaded sealed and unsealed specimens.

5.2.2 Inferred strains

No provision was made in this project for the measurement of the actual autogenous shrinkage strains of the sealed samples. Therefore, the autogenous shrinkage strains were inferred from the test results of one of the mixes used in a research project conducted by Alexander (1994b). These results are presented in

Figure 2.3 and mix 2.1 was selected as the reference mix because it comprised OPC as well as a water content of 190 l/m^3 which is of similar magnitude to that used for all the mixes in this research (195 l/m^3). In addition, the cement content, w/c ratio and a/c ratio of Alexander's reference mix are of a similar magnitude to the corresponding mix proportions of the three low strength mixes (Q1, G1 and A1) in this project. The abovementioned similarities are evident from a comparison of the mix proportions given in Tables 2.1 and 4.1.

For each of the six mixes included in this project, the autogenous shrinkage strains, corresponding to the times at which total strains and drying shrinkage strains were measured, were calculated by considering the a/c ratio of the mix relative to the a/c ratio of Alexander's reference mix (by mass) which was 5.12. The premise was adopted that if the a/c ratio of a mix being considered (from this project) is higher than 5.12 then the expected autogenous shrinkage at any age should be relatively lower than that of Alexander's reference mix at the same age. This premise is based on the fact that autogenous shrinkage occurs in the paste and should increase with an increase in cement content relative to the aggregate content, for a constant water content. The inferred autogenous shrinkage values were calculated using the equations below (which assume a linear relationship) and are shown in Table E.1 in the appendix.

$$F = 5.12/(a/c) \tag{5.1}$$

$$\varepsilon_{sha}(t) = F \times \varepsilon_{shR}(t) \tag{5.2}$$

where,

F	=	Factor accounting for the relative difference between the a/c ratios of Alexander's reference mix (5.12) and the mix being considered
a/c	=	aggregate cement ratio of mix considered
ε_{shR}	=	Autogenous shrinkage strain of Alexander's reference mix at time t (t = 0, 10, 30, 100 or 200 days), determined from Figure 2.3
ε_{sha}	=	Autogenous shrinkage strain of the mix under consideration at time t.

The autogenous shrinkage strains corresponding to the times at which total strains and drying shrinkage strains were measured in this project were determined by linearly interpolating between the ε_{sha} values at times t = 0, 10, 30, 100 or 200 days.

The cumulative autogenous shrinkage strains with time inferred for the mixes with a w/c ratio of 0.56 (Q1, G1, A1) and for the mixes with a w/c ratio of 0.4 (Q2, G2, A2) are shown in Figures 5.2 and 5.3.

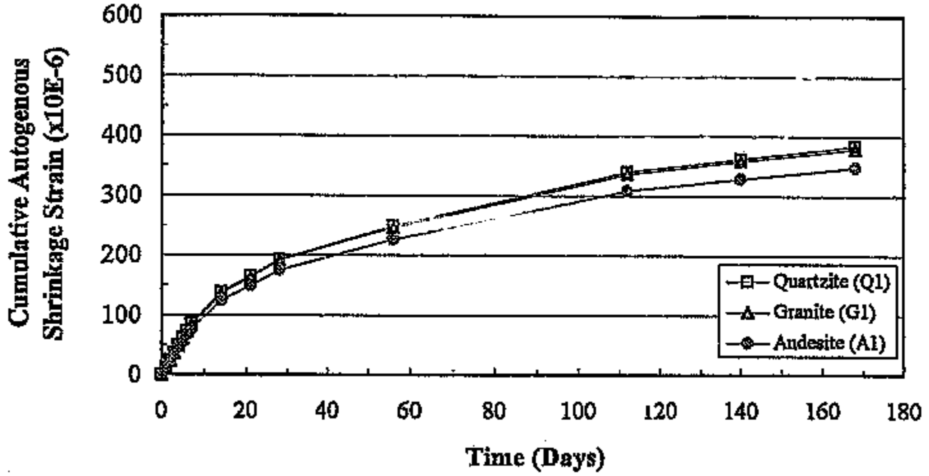


Figure 5.2 Cumulative autogenous shrinkage strain versus time for mixes with a w/c ratio of 0.56

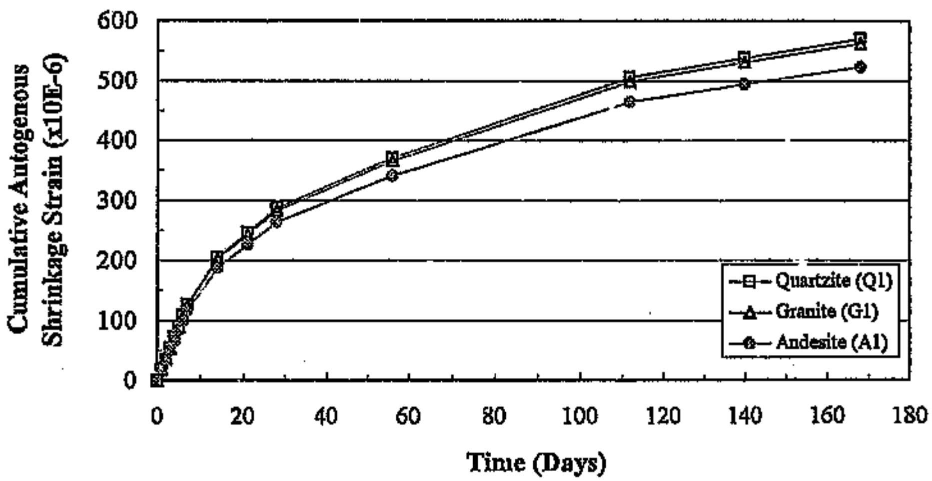


Figure 5.3 Cumulative autogenous shrinkage strain versus time for mixes with a w/c ratio of 0.4

As the shrinkages in Figures 5.2 and 5.3 were inferred on the basis of their a/c ratio, they cannot be used to assess the actual effect of the aggregate type on autogenous shrinkage. These figures indicate the following intended trends:

- the low w/c ratio mixes (0.4) have higher shrinkage values than the high w/c ratio mixes (0.56) as the former contain more cement, and
- in the case of the mixes of each of the w/c ratios, the higher the a/c ratio, the lower the autogenous shrinkage strain.

5.3 Calculation of Creep Strains and Creep-Time Functions

For each of the six mixes included in this project, the measured strains were used to calculate the total creep strains (sum of basic plus drying), specific total creep, total creep coefficients, basic creep strains, specific basic creep and basic creep coefficients, pertaining to the times at which measurements were taken, which are presented in Tables 5.1 to 5.3.

The average cumulative total creep ($\varepsilon_{c\,(tot)}(t)$) and basic creep ($\varepsilon_{c\,(bas)}(t)$) strains were calculated from equations 5.3 and 5.4 (below), respectively. The specific creep (C_o) and creep coefficient (ϕ) values were calculated by applying equations 2.3 and 2.4, respectively.

Table 5.1 Results of creep measurements on quartzite-aggregate concretes

Mix No.	Q1						Q2					
Applied Stress	9.25 MPa						16.30 MPa					
	Total Creep (sum of basic and drying)			Basic Creep			Total Creep (sum of basic and drying)			Basic Creep		
Age (Days)	Average Cumulative Strain (x10E-6)	Specific Creep (x10E-6/ MPa)	Creep Coefficient (Creep Factor)	Average Cumulative Strain (x10E-6)	Specific Creep (x10E-6/ MPa)	Creep Coefficient (Creep Factor)	Average Cumulative Strain (x10E-6)	Specific Creep (x10E-6/ MPa)	Creep Coefficient (Creep Factor)	Average Cumulative Strain (x10E-6)	Specific Creep (x10E-6/ MPa)	Creep Coefficient (Creep Factor)
1	61	6.620	0.161	85	9.234	0.252	91	5.605	0.185	85	5.194	0.182
2	111	12.036	0.293	73	7.937	0.217	125	7.654	0.252	78	4.773	0.167
3	128	13.841	0.337	38	4.124	0.113	155	9.533	0.314	88	5.376	0.188
4	175	18.957	0.461	7	0.721	0.020	186	11.411	0.376	56	3.418	0.120
5	178	19.258	0.469	22	2.432	0.066	203	12.436	0.409	51	3.106	0.109
6	186	20.160	0.491	27	2.941	0.080	205	12.606	0.415	63	3.880	0.136
7	223	24.072	0.586	21	2.245	0.061	242	14.826	0.488	65	3.971	0.139
14	320	34.604	0.842	23	2.533	0.069	353	21.656	0.713	51	3.113	0.109
21	392	42.427	1.033	85	9.243	0.252	414	25.413	0.837	113	6.916	0.242
28	423	45.737	1.113	94	10.128	0.276	437	26.779	0.882	133	8.157	0.285
56	576	62.286	1.516	185	20.022	0.546	562	34.463	1.135	224	13.762	0.481
112	715	77.332	1.882	246	26.541	0.724	662	40.610	1.337	254	15.554	0.544
140	757	81.845	1.992	304	32.888	0.897	709	43.513	1.433	302	18.543	0.649
168	799	86.359	2.102	307	33.218	0.906	745	45.733	1.506	298	18.287	0.640

Table 5.2 Results of creep measurements on granite-aggregate concretes

Mix No.	G1						G2					
Applied Stress	9.42 MPa						16.30 MPa					
	Total Creep (sum of basic and drying)			Basic Creep			Total Creep (sum of basic and drying)			Basic Creep		
Age (Days)	Average Cumulative Strain (x10E-6)	Specific Creep (x10E-6/ MPa)	Creep Coefficient (Creep Factor)	Average Cumulative Strain (x10E-6)	Specific Creep (x10E-6/ MPa)	Creep Coefficient (Creep Factor)	Average Cumulative Strain (x10E-6)	Specific Creep (x10E-6/ MPa)	Creep Coefficient (Creep Factor)	Average Cumulative Strain (x10E-6)	Specific Creep (x10E-6/ MPa)	Creep Coefficient (Creep Factor)
1	145	15.354	0.449	52	5.471	0.144	156	9.554	0.284	82	5.031	0.141
2	178	18.900	0.553	40	4.197	0.111	214	13.140	0.391	78	4.785	0.134
3	200	21.263	0.622	41	4.400	0.116	256	15.701	0.467	93	5.706	0.160
4	239	25.400	0.743	71	7.558	0.199	273	16.726	0.498	117	7.178	0.202
5	245	25.991	0.760	45	4.807	0.127	298	18.263	0.543	93	5.706	0.160
6	275	29.241	0.855	42	4.420	0.117	320	19.629	0.584	93	5.706	0.160
7	289	30.718	0.899	38	4.032	0.106	345	21.166	0.630	83	5.092	0.143
14	376	39.878	1.167	69	7.376	0.195	437	26.801	0.797	118	7.239	0.203
21	429	45.492	1.331	123	13.079	0.345	506	31.070	0.924	191	11.718	0.329
28	479	50.810	1.486	156	16.607	0.438	554	33.972	1.010	215	13.190	0.371
56	576	61.152	1.789	226	23.958	0.632	665	40.803	1.214	318	19.509	0.548
112	696	73.857	2.161	340	36.079	0.952	793	48.657	1.447	397	24.356	0.684
140	737	78.289	2.290	379	40.244	1.062	835	51.219	1.523	412	25.276	0.710
168	760	80.653	2.359	389	41.265	1.089	846	51.902	1.544	422	25.890	0.728

Table 5.3 Results of creep measurements on andesite-aggregate concretes

Mix No.	A1						A2					
Applied Stress	12.00 MPa						18.47 MPa					
	Total Creep (sum of basic and drying)			Basic Creep			Total Creep (sum of basic and drying)			Basic Creep		
Age (Days)	Average Cumulative Strain (x10E-6)	Specific Creep (x10E-6/ MPa)	Creep Coefficient (Creep Factor)	Average Cumulative Strain (x10E-6)	Specific Creep (x10E-6/ MPa)	Creep Coefficient (Creep Factor)	Average Cumulative Strain (x10E-6)	Specific Creep (x10E-6/ MPa)	Creep Coefficient (Creep Factor)	Average Cumulative Strain (x10E-6)	Specific Creep (x10E-6/ MPa)	Creep Coefficient (Creep Factor)
1	120	9.965	0.390	25	2.088	0.072	131	7.093	0.291	69	3.750	0.153
2	164	13.676	0.535	47	3.954	0.136	173	9.354	0.384	98	5.295	0.216
3	175	14.604	0.571	39	3.269	0.112	176	9.505	0.390	81	4.375	0.178
4	209	17.388	0.680	39	3.281	0.113	212	11.464	0.471	90	4.865	0.198
5	225	18.779	0.734	42	3.524	0.121	228	12.368	0.508	90	4.848	0.198
6	231	19.243	0.752	42	3.535	0.122	242	13.121	0.539	89	4.832	0.197
7	256	21.331	0.834	56	4.706	0.162	273	14.779	0.607	93	5.021	0.205
14	370	30.840	1.205	93	7.747	0.266	401	21.711	0.891	138	7.452	0.304
21	448	37.335	1.459	134	11.147	0.383	482	26.081	1.070	183	9.915	0.404
28	526	43.829	1.713	162	13.471	0.463	551	29.848	1.225	206	11.173	0.456
56	693	57.746	2.257	269	22.442	0.772	718	38.890	1.596	321	17.402	0.710
112	846	70.503	2.756	402	33.468	1.151	866	46.877	1.924	406	21.991	0.897
140	882	73.518	2.874	479	39.919	1.373	921	49.891	2.048	458	24.791	1.011
168	924	76.997	3.010	495	41.268	1.419	955	51.699	2.122	462	25.029	1.020

$$\varepsilon_{c\,(tot)}(t) = \varepsilon(t) - \varepsilon_e - \varepsilon_{shd}(t) \tag{5.3}$$

$$\varepsilon_{c\,(bas)}(t) = \varepsilon(t) - \varepsilon_e - \varepsilon_{sha}(t) \tag{5.4}$$

where,

$\varepsilon_{c\,(tot)}(t)$	=	total creep strain at any time t
$\varepsilon_{c\,(bas)}(t)$	=	basic creep strain at any time t
$\varepsilon(t)$	=	total concrete strain at any time t
ε_e	=	average instantaneous elastic strain recorded immediately after loading
$\varepsilon_{shd}(t)$	=	cumulative drying shrinkage strain at any time t (recorded on the companion specimens)
$\varepsilon_{sha}(t)$	=	autogenous shrinkage strain at any time t (inferred as described in paragraph 5.2.2)

5.4 Drying Shrinkage Strains

A detailed discussion of shrinkage strains is beyond the scope of this research. However, as the shrinkage strain magnitudes were used in the calculation of the creep strains in this research, it appears relevant to comment on the shrinkage exhibited by the companion samples. The average cumulative drying shrinkage strain with time measured on the companion specimens of mixes with a w/c ratio of 0.56 (Q1, G1, A1) and those measured on the specimens with a w/c ratio of 0.4

(Q2, G2, A2) are shown in Figures 5.4 and 5.5, which are plotted to the same scale. The negative shrinkage values which were recorded for the G1 specimens within the first two days of drying (shown in Table D.3 in the appendix) are not included in Figure 5.4.

With reference to Figures 5.4 and 5.5, it is evident that, for each aggregate type, the specimens with the higher w/c ratio (0.56) exhibited less drying shrinkage than those with the lower w/c ratio (0.4). In addition, it was noted that the difference between the shrinkage of the two strength grades of each aggregate type appears to reduce with time and the rate of shrinkage of all the mixes decreases with time. These trends are in agreement with the findings of Alexander (1993b).

The decrease in cumulative shrinkage at an age of 168 days (after loading), which is most pronounced for the quartzite concretes, is probably attributable to the increase in both relative humidity and temperature (see Tables D.1 to D.6) which resulted from a temporary breakdown of the air conditioner during the week in which those shrinkage strains were recorded.

Furthermore, the concretes containing quartzite aggregate displayed less shrinkage than both the granite and andesite concretes, for both w/c ratios. The specimens containing granite generally exhibited less shrinkage than those containing andesite in the case of the high w/c ratio, but more shrinkage than the andesite in the comparison of specimens with the lower w/c ratio.

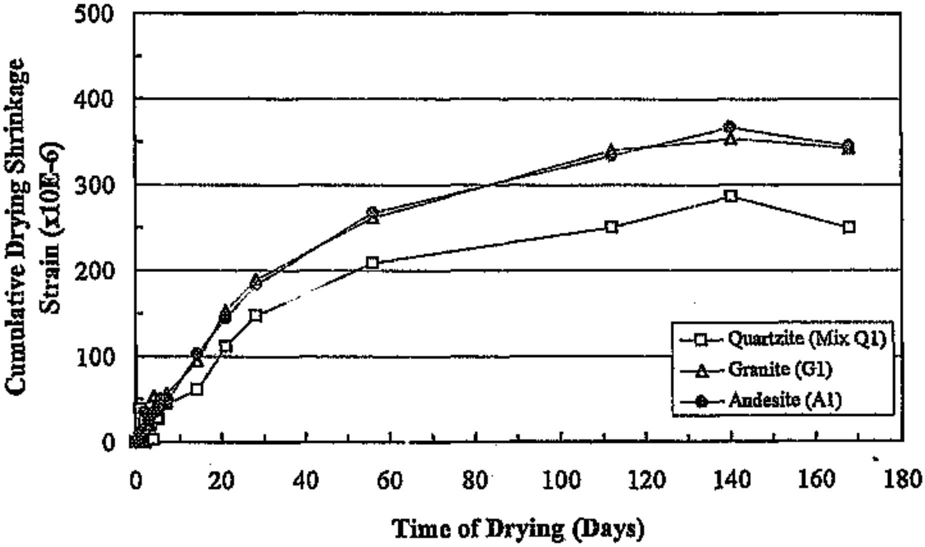


Figure 5.4 Cumulative drying shrinkage strain versus time of drying for shrinkage specimens with a w/c ratio of 0.56

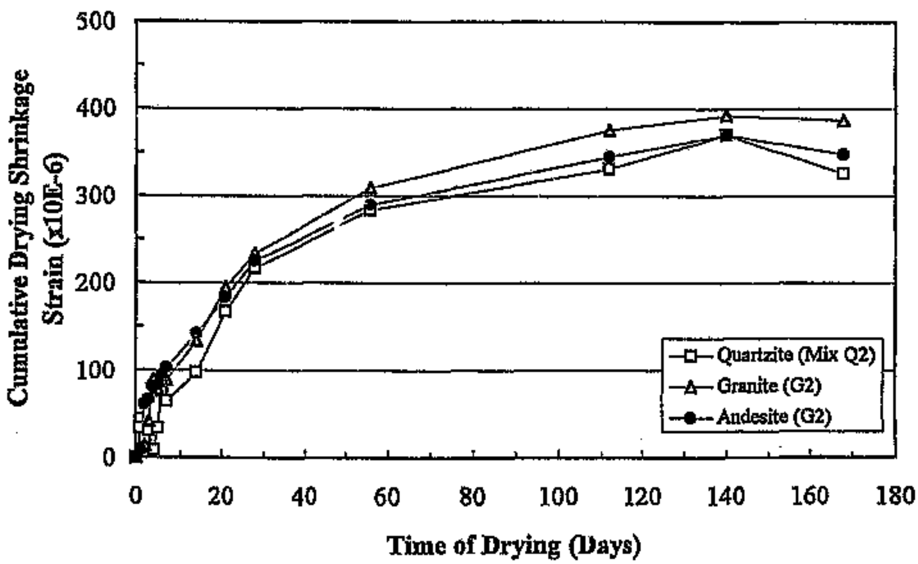


Figure 5.5 Cumulative drying shrinkage strain versus time of drying for shrinkage specimens with a w/c ratio of 0.4

Extensive shrinkage tests carried out by Davis and Alexander (1992) on concretes with aggregates from the same sources as those used in this project showed the relative shrinkage of the concrete containing granite to be higher than that of concrete containing quartzite but lower than andesite concrete. This order of relative shrinkages with the use of different aggregates is indicated in Figure 5.4. Nevertheless, according to Davis and Alexander (1992), the relative shrinkages are merely intended for general guidance as the shrinkage of concretes containing aggregates from a particular source can vary.

5.5 Creep

5.5.1 Correlation of total creep with E of aggregate

The measured elastic moduli, ranges and averages for each of the three aggregate types (determined in this research) are shown in Table 5.4. This table also includes the ranges and average values determined by Davis and Alexander (1992) for the same aggregates from the same sources.

For the purposes of comparing the influence of aggregate alone on specific total creep, the specific total creep values at 168 days (six months) after loading were modified to account for the different w/c ratios.

Table 5.4 Results of elastic moduli tests on cores

Aggregate Type	Elastic Moduli of Rock Cores					
	Measured				Davis and Alexander	
	Core Number	E (GPa)	Range (GPa)	Mean (GPa)	Range (GPa)	Mean (GPa)
Quartzite (Ferro)	1/1	73.1				
	1/2	87.6	59 - 88	73	42 - 98	70
	1/3	59.4				
Granite (Jukskei)	1/1	65.8				
	1/2	79.5				
	1/3	68.6				
			66 - 80	70	27 - 93	60
	2/1	66.7				
	2/2	70.5				
	2/3	70.3				
Andesite (Eikenhof)	1/1	82.4				
	1/2	93.5	82 - 94	89	80 - 110	95
	1/3	91.3				

This modification, which is similar to one carried out by Davis and Alexander (1992), entailed the adjusting of the specific total creep values by the ratio of their compressive strength at the age of loading to the mean of the compressive strengths of all six mixes (54.5 MPa). The average of the two adjusted specific creep values for each aggregate type was then expressed as a ratio of the mean of the six adjusted values (61.549 x 10E-6/MPa), to obtain a relative creep value for each aggregate type. The adjustment factors, adjusted specific total creep values and relative creep factors are given in Table F.1 in the appendix which includes the relative creep values determined by Davis and Alexander (1992) for the same aggregates.

Figure 5.6 shows a correlation of the relative creep with average elastic modulus of the aggregate using the specific total creep results from this research and from the work by Davis and Alexander (1992). The letters Q,G and A denote quartzite, granite and andesite concretes, respectively. The results in Figure 5.6 indicate significant variations in the stiffness of aggregates from a particular source. Furthermore, the higher the average elastic modulus of an aggregate, the more the relative creep of the concrete. The regression equations and correlation coefficients applicable to the values from this research and from Davis and Alexander's (1992) research, when considered separately and together, are given in Table 5.5.

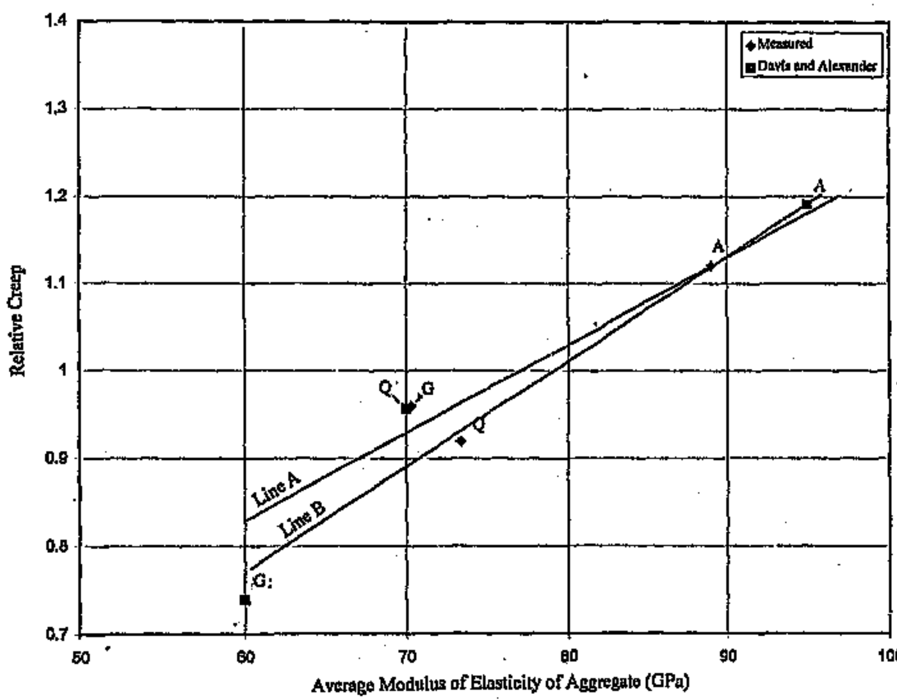


Figure 5.6 Relationship between relative creep and elastic modulus of aggregates

Table 5.5 Statistics pertaining to the correlation of relative creep to average modulus of elasticity of aggregates

Line	Data Source	Regression Equation	r
A	Measured	y = 0.010x + 0.229	0.941
B	Davis and Alexander	y = 0.012x + 0.052	0.973
	Combined	y = 0.012x + 0.092	0.965

The correlations in Figure 5.6 show an opposite trend to those established by Rusch et al., (1962) and The Concrete Society (1974) (see Figure 2.11) which indicate that the higher the elastic modulus of the aggregate, the greater the restraint offered by the aggregate to the creep of the paste. However, the work of the abovementioned researchers shows creep of concrete to be relatively insensitive to aggregate stiffness in the case of aggregates with a modulus of elasticity in excess of approximately 70 GPa. Therefore, the correlations shown in Figure 5.6 do not disprove the trends which were established by other researchers on the basis of an aggregate stiffness range greater than that used in this research.

From the above, it is concluded that some other property of the aggregate, rather than stiffness, has a more significant effect on creep.

5.5.2 Total creep strains

Since the stress:strength ratio for all the mixes was constant (25 per cent), creep in this case should be independent of w/c ratio and applied stress if the differences in the volumetric paste volumes are accounted for (Orchard, 1979). However, in this research, the creep results were not normalised to account for the different paste volumes, which were 0.31 and 0.35 of the mix volume in the case of the low strength and high strength concretes, respectively. The reason for this is that the creep of the paste which is required for this adjustment was not determined. Therefore, specific creep strains were used when comparing the creep exhibited by the different mixes. The specific total creep (basic and drying) values measured on the prisms of each of the six mixes since the time of loading are shown in Figure 5.7.

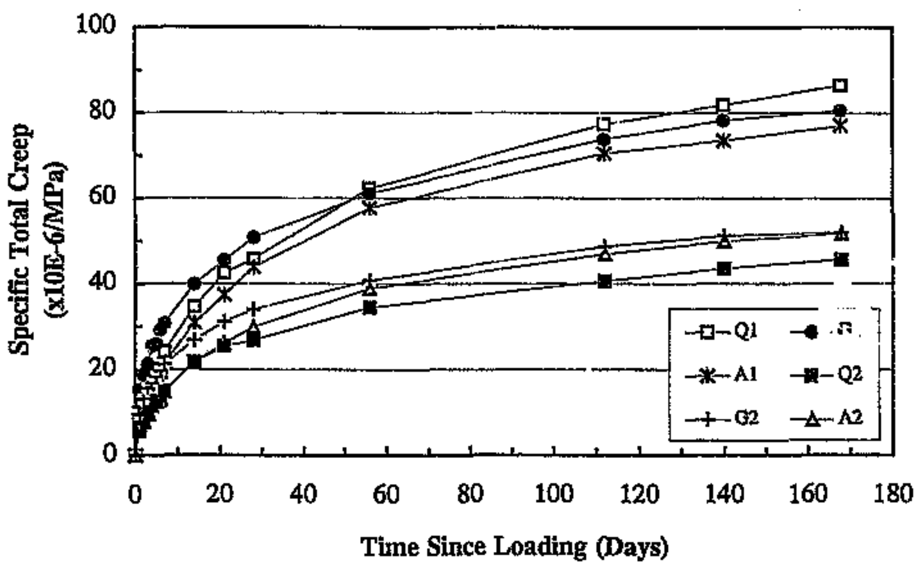


Figure 5.7 Specific total creep versus time since loading

It is evident from Figure 5.7 that, for each of the aggregate types, the mix with the lower w/c ratio (stiffer mix) yielded a lower specific total creep value. This is in accordance with the findings of Reutz (1965), Ballim (1983), Smadi et al., (1987), Addis (1992) and Fiorato (1995).

The reason for the abovementioned trend is that the concrete with the higher strength and stiffness has a relatively lower porosity of the hardened cement paste matrix in comparison with the lower strength concrete (Muller and Kuttner, 1996). Furthermore, the curves of the higher w/c ratio (0.56) mixes indicate that the order of increasing specific total creep of concrete, for most of the test period, with the use of the different aggregates, to be andesite, granite and quartzite. By relative comparison, the positions of the specific total creep curves of the lower w/c ratio (0.4) mixes differ in that the quartzite concretes yielded the lowest specific total creep values. Hence, when considering the average elastic moduli values of the three aggregate types, which are given in Table 5 4, it is evident that no correlation exists between the specific total creep of the concrete and the stiffness of the aggregate included in the concrete.

Research conducted by Davis and Alexander (1992) on creep of concretes with various aggregates, including those used in this research, showed concrete creep with the use of these aggregates to increase in the order granite, quartzite and andesite. Referring to Figure 5.7, it is evident that the positions of the specific total creep curve of the andesite concretes (in the case of the higher w/c ratio) and the granite concretes (in the lower w/c ratio) are in disagreement with the results of Davis and Alexander (1992).

5.5.3 Basic creep strains

The specific b..sic creep of each of the six mixes since the time of loading are shown in Figure 5.8.

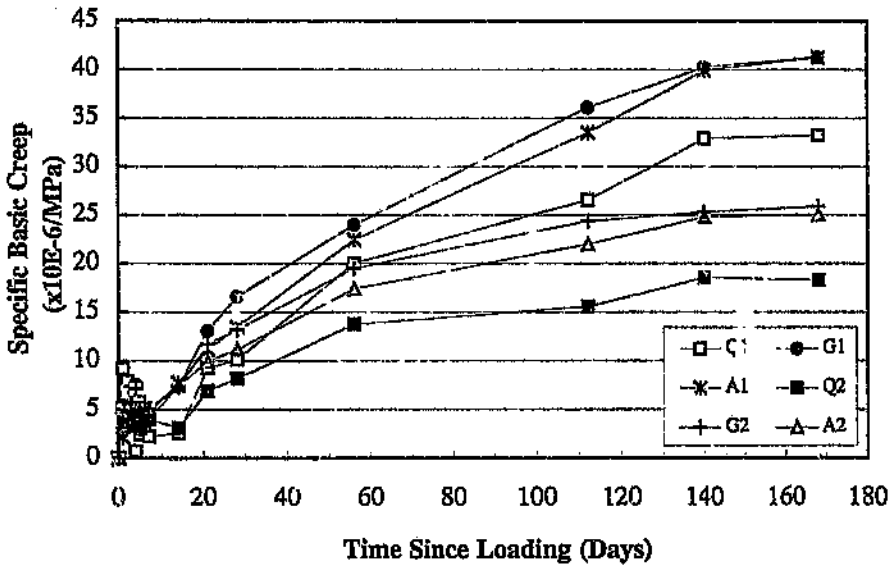


Figure 5.8 Specific basic creep versus time since loading

It is evident from Figure 5.8 that, for each of the aggregate types, the mix with the lower w/c ratio yielded a lower specific basic creep value. This trend was also observed in the specific total creep results (above). A comparison of the specific basic creep curves indicates the order of increasing specific basic creep with the use of different aggregates to be quartzite, andesite and granite in the case of both w/c ratios. This trend indicates that, as was the case in the specific total creep results, no correlation exists between the specific basic creep of concrete and the stiffness of the aggregate included in the concrete. Furthermore, the relative positions of the curves in Figure 5.8 differ from the findings of Davis and Alexander (1992) which indicate the total creep with the use of these aggregates to increase in the order granite, quartzite and andesite.

A comparison of the relative positions of the specific total creep and specific basic creep curves for each of the w/c ratios (Figures 5.7 and 5.8), indicates only one difference. This difference is the position of the quartzite curve in the higher w/c ratio mixes which represents the highest and lowest values in the case of the specific total creep and specific basic creep, respectively. Note that the specific basic creep values of the granite and andesite concretes of each w/c ratio are of a similar magnitude.

It should be borne in mind that the measured specific basic creep values incorporate the magnitude of the inferred autogenous shrinkage strain. The autogenous shrinkage strains were inferred from data by Alexander (1994b), pertaining to concrete containing only quartzite aggregates, which were of magnitudes up to three times higher than anticipated (Alexander, 1998). Therefore, the accuracy of the specific basic creep values in this research has not been verified.

5.5.4 Comparison of total and basic creep

The specific total creep and specific basic creep strains of the six mixes exhibited over the six month loading period are shown in Figures G.1 to G.6 in the appendix.

For each of the mixes, for the loading period considered, the specific basic creep was expressed as a percentage of the specific total creep at each of the ages at which measurements were taken. The average of these ratios for each mix are given in Table 5.6.

Table 5.6 Average of specific basic creep to specific total creep ratios (expressed as a percentage) for the six month loading period

Average Specific Basic/Total Creep (%)					
Mix Q1	Mix Q2	Mix G1	Mix G2	Mix A1	Mix A2
25.7	39.2	30.9	39.7	31.3	43.3

The average specific basic/specific total creep ratio given for mix Q1 in the table above excludes the data pertaining to one day after loading, as the sealed samples of this mix indicated the specific basic creep to exceed the specific total creep at an age of one day after loading by approximately 39 per cent. This observation, which contradicts the assumed fact that drying always enhances creep, is due to swelling deformation of the sealed concrete. The swelling is due to the release of surface tension of capillary water due to the change of vapour pressure above water menisci in the capillaries (Kovler, 1996). This swelling usually takes place if the concrete is subjected to drying prior to sealing.

From the data in Table 5.6 it appears that, for the concrete of each aggregate type, the ratio of the average specific basic to total creep is larger in the case of the lower w/c ratio mix. This indicates that the drying creep, ε_{fd} (t) in equation 2.2, is less in the case of the lower w/c ratio mixes.

According to Muller and Kuttner (1996) this is due to the relatively low porosity of the hardened cement paste of the high strength concrete, in comparison with the low strength concrete, which affects (reduces) the diffusion properties of the concrete. In addition, in the case of both w/c ratios, the drying creep of the concretes comprising the three different aggregates increases in the order andesite, granite and quartzite.

5.5.5 Correlation of secant modulus of concrete and creep strain

The relationship between the specific creep values at 168 days and secant modulus of each concrete mix at the age of loading (28 days) is shown in Figure 5.9.

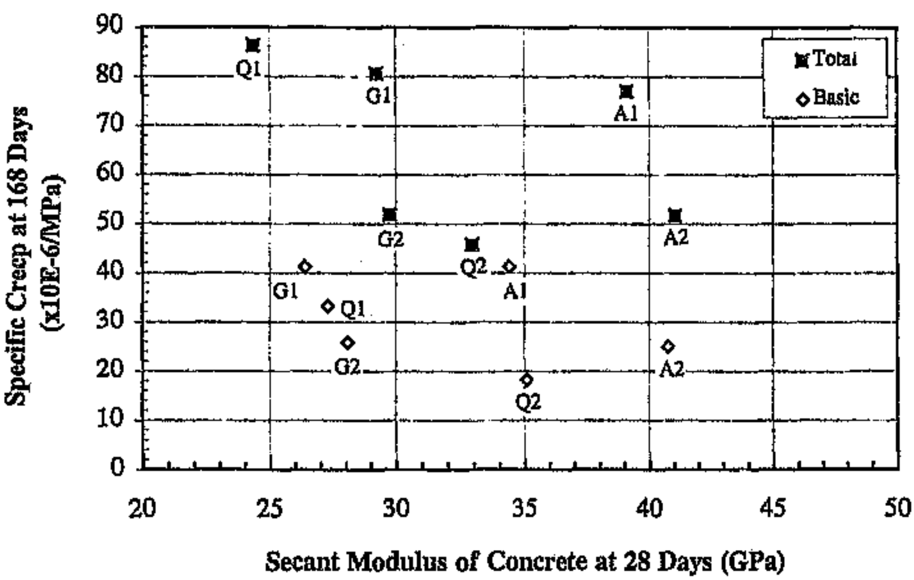


Figure 5.9 Relationship between specific creep at 168 days and secant modulus of concrete at 28 days

With reference to Figure 5.9, it is evident that no correlation exists between the secant moduli and the specific creep of any of the mixes used. Nevertheless, the secant moduli used in Figure 5.9 are not accurate, as a difference of one division on the Demec gauge (which is equal to 17 micro-strain) could alter these values by a difference ranging from 0.89 to 2.05 GPa.

5.6 Conclusions

5.6.1 Shrinkage strains

The results of this research indicate that, for each aggregate type, the specimens with the higher w/c ratio (0.56) exhibited less drying shrinkage than those with the lower w/c ratio (0.4). Furthermore, no correlation was established between the drying shrinkage magnitude and the stiffness of the included aggregate.

The autogenous shrinkage strain values used in this research were inferred from data by Alexander (1994b). The accuracy of the inferred values was not determined. Hence, further research is justified to establish the actual autogenous shrinkage strain magnitudes for the mixes used in this project.

5.6.2 Effect of aggregate stiffness on creep strain

At any age after loading, the specific basic creep values (for both w/c ratios) and specific total creep values for the lower w/c ratio mixes, with the use of the different aggregates included in this research, increase in the order quartzite, andesite and granite. In the case of the specific total creep for the higher w/c ratios, the values of the concretes made with different aggregates increase in the order andesite, granite and quartzite. These results indicate that no correlation exists between the creep of concrete and the stiffness of the aggregate included in the concrete. Furthermore, the correlation between relative creep and aggregate stiffness indicated that the creep of concrete increased with increasing stiffness of the included aggregate. This trend is opposite to correlations established by Rusch et al., (1962) and The Concrete Society (1974), on the basis of an aggregate stiffness range greater than that used in this research. However, the abovementioned relationships indicate the creep of concrete to be relatively insensitive to aggregate stiffness in the case of aggregates with a modulus of elasticity in excess of approximately 70 GPa.

The abovementioned conclusions appear to be attributable to the stress strain behaviour of the aggregate/paste interfacial zone. Consideration of the behaviour of the interfacial zone, which is dependent on bond strength and density of this zone, is of significance and should not be overlooked by assuming that concrete is a two phase material (Nilsen and Monteiro, (1993), Alexander, (1991), Alexander and Davis (1992), Alexander (1993b) and Mindess and Alexander (1995)). Therefore, in this research the measured creep that was assumed to have taken place in the paste may have included the magnitude of strains which occurred at the interfacial zone. This important phenomenon was reiterated by Alexander (1997). According to Alexander and Milne (1995), the size of the interfacial region and perhaps the bonding of the aggregate to the matrix may be influenced by the hydrophobic or hydrophilic nature of aggregates.

5.6.3 Comparison of results with those of Davis and Alexander (1992)

The relative magnitudes of concrete creep with the use of different aggregates, obtained for the mixes in this research (see Section 5.6.2), are in disagreement with the outcome of research conducted by Davis and Alexander (1992) on creep of concretes with various aggregates, including those used in this research, which shows concrete creep with the use of these aggregates to increase in the order granite, quartzite and andesite. These disagreements appear to be mainly due to the possible inaccuracy of the findings of Davis and Alexander (1992), resulting from a number of data adjustments, and to a lesser extent due to variations in the properties of aggregates from the same source, as discussed below.

Adjustments to data used by Davis and Alexander (1992)

The creep data analysed by Davis and Alexander (1992) was applicable to concretes with aggregates from 23 different sources which had different test variables and hence a number of adjustments were made to the data before it could be compared directly in order to establish the relative creep values. These included:

- adjustments to account for different ages of loading, w/c ratios and cement types

- extrapolation to obtain the creep at an age of five years after loading
- normalisation to account for effects due to differences in paste volumes
- adjustment of creep values such that their mean was equal to the value predicted using the BS 8110 method at 5 years, assuming a common age of first loading of one year
- the relative creep of concretes of each aggregate type was determined relative to the mean.

Variations in aggregates from the same source

Generically similar aggregates from different geographical regions, within South Africa, and also within a particular quarry may vary in their mineralogical, petrological and physical properties (Alexander, 1990).

5.6.4 Effect of secant modulus of concrete at age of loading on creep

For all the mixes considered, no correlation was established between the secant modulus of the concrete at the age of loading (28 days) and the specific creep values at 168 days after loading.

CHAPTER 6

COMPARISON OF MEASURED AND PREDICTED CREEP STRAINS

6.1 Introduction

The creep prediction methods reviewed in Chapter 3 were used to predict the specific total creep (basic and drying creep) as well as the specific basic creep strains, at the same ages at which measurements were taken (see Section 4.4.2), for the concrete of each of the six mixes used in this investigation. The values of the intrinsic and extrinsic factors pertaining to the test specimens and their environment were used as the values for the relevant factors required as input by the different prediction methods. The measured and predicted values are compared and discussed in Section 6.4. These results are in turn compared with the results of other research projects where experimental and predicted creep values were compared, in Section 6.5.

The measured specific total creep and specific basic creep values recorded on the concrete specimens of each of the six mixes since the time of loading are shown in Tables 5.1 to 5.3 in Chapter 5.

The empirical factors as well as other variables used in each prediction method are given in Tables H.1 to H.18 in Appendix H. The measured and predicted total creep coefficients and basic creep coefficients, for different times under load, for each of the six mixes are given in Tables I.1 to I.6 in Appendix I. The measured and predicted specific total creep and specific basic creep values, for the concrete of each strength grade for each of the aggregate types investigated, are given in Tables J.1 to J.6 in Appendix J.

Specific creep values were used for comparative purposes and not creep coefficient (ϕ) values. The reason for this is that the ϕ value and elastic modulus (E) value are both considered in the calculation of the predicted creep in all the design code models included in this investigation and the different models do not all prescribe the same equation for the estimation of E.

6.2 Assumptions Made in Analysis

6.2.1 General assumptions

Absence of reinforcement

All the prediction methods used are applicable to plain (unreinforced) concrete. It has however been shown that the creep in concrete members with symmetrically distributed reinforcement is relatively less than that of identical unreinforced members (Troxell et al., 1958 and Lambotte et al., 1983).

Elastic moduli values of concrete

With the exception of the RILEM Model B3 (1995), all the models reviewed in Chapter 3 express creep strain as the product of the elastic deformation of the concrete (usually at the time of loading) and the creep coefficient. Therefore, the magnitude of the elastic modulus of the concrete has a significant influence on the predicted creep values.

The elastic moduli used in the creep predictions were determined in accordance with each method. It should be noted that the AS 3600 (1988) method also makes provision for the use of an elastic modulus of the concrete determined by testing.

The BS 8110 (1985) method specifies the use of only characteristic strength as the basis for determining elastic modulus. The CEB-FIP (1978) and CEB-FIP (1990) methods make provision for the determination of the elastic moduli on the basis of characteristic or actual concrete strength. In this investigation, the mean value of the actual compressive strength of the concrete prisms of each mix, at 28 days, was used for the prediction of elastic moduli.

With the exception of the BS 8110 (1995) method, all the creep prediction methods included in this investigation prescribe that the 28 day compressive strength of the concrete, used to calculate the elastic modulus (E_c), should be determined on cylinders. Therefore, where the cylinder strengths were required, the 28 day compressive strength values determined on 150 mm cubes were reduced to their equivalent cylinder strengths in accordance with the conversions given in Table 3.5, which derives from the CEB-FIP (1990) Model Code.

Relative humidity and temperature values

In the calculation of specific total creep, using methods that account for the relative humidity of the ambient environment, variations in relative humidity were ignored and a constant value of 65 per cent was assumed throughout. The reason for this is that the relative humidity in the creep testing room was maintained at 65±5 per cent and was only recorded at times when readings were taken. Therefore, it would be incorrect to assume that the relative humidity values that were recorded when readings were taken prevailed during the periods between readings. A relative humidity of 100 per cent was used in the basic creep predictions. Where the actual temperature was required by a prediction method, a temperature of 23 °C was used.

6.2.2 Method specific assumptions

Assumptions and comments specific to each method used are discussed below.

BS 8110 (1985)

The elastic moduli were determined using equation 3.4 and design values (Tables A.1 and A.2) which were established by Davis and Alexander (1992).

The effective section thickness of the test specimens was 50 mm. Therefore, the lowest section thickness of 150 mm was used when determining the creep coefficient for total creep from Figure 3.1.

As mentioned in Chapter 3, this method enables the estimation of the final (30 year) creep strain (ε_{cc}) of which 40 %, 60 % and 80 % may be assumed to develop during the first month, 6 months and 30 months of loading, respectively. Therefore, in order to predict the creep strain at the ages at which measurements were taken on the samples included in this investigation, equation 6.1 which was developed by Marques (1992) was used. This equation was obtained from a linear regression of % creep and log (t-τ).

$$\% \text{ of final creep} = 100[0.257906(\log_{10}(t-\tau)) + 0.028622] \text{ for } (t-\tau) \geq 1 \text{ day} \qquad (6.1)$$

where,

(t-τ) = age since loading (days)

Two sets of creep values were predicted for each of the mixes, one excluding and the other including the relative creep coefficients derived by Davis and Alexander (1992). In this chapter, the former and latter procedures are referred to as the BS 8110 (1985) method and the BS 8110 (1985) - Modified method, respectively.

ACI 209 (1992) method

As the actual air content in the samples tested was not known, the correction factor for air content (γ_6) was assumed to be equal to the minimum allowable value of 1.0. It should be noted that a value of 1.0 is yielded for air contents less than eight per cent. In any event, these correction factors are normally not excessive and in most cases tend to offset each other, they are therefore often neglected for design purposes (ACI 209, 1992).

AS 3600 (1988)

This method is intended to apply to structures with a characteristic compressive strength at 28 days within the range of 20 to 50 MPa. For this reason, this method was only applied to predict creep strains for various ages after loading for the low strength grade concretes of the aggregate types included in the investigation (i.e mixes Q1, G1 and A1).

The basic creep factors were obtained by linearly interpolating between the values given in Table 3.3. For the purposes of the basic creep predictions, the hypothetical thickness (t_h) used for the determination of the creep factor coefficient (k_2) was assumed to be the maximum of 400 mm.

CEB-FIP Model Code (1978)

The CEB-FIP (1978) method prescribes an adjustment to the age at which the prediction is made in cases where the curing temperature of the concrete is significantly different from 20 °C. This adjustment was not implemented as the concrete was cured at a temperature of 22 ± 1 °C.

CEB-FIP Model Code (1990)

When calculating the notional creep coefficient (ϕ_0), the age of concrete at loading (t_0) substituted into equation 3.52 to determine $\beta(t_0)$ was not adjusted to account for the effect of elevated or reduced temperatures on the maturity of the concrete, as the prevailing temperature was assumed constant for all the predictions. Furthermore, the value of t_0 was unaffected by the second possible adjustment which accounts for the effect of cement type.

RILEM Model B3 (1995)

During the application of this method the cross-section shape factor (k_s) for the test prisms was taken to be that prescribed for an infinite square prism (1.25).

6.3 Statistical Techniques Employed

6.3.1 T-Test

The paired (two-tailed) t-Test was applied to paired data values to determine whether the two samples are likely to have come from the same two underlying populations that have the same mean (Moroney, 1984, Cohen, 1991 and Spiegel, 1992). Where applied, the null hypothesis was assumed (i.e. any observed differences are due to fluctuations within the same population). The five percent significance level was decided upon.

The probability calculated is associated with the Student's t-Test. The significance levels established (P) indicate the probability that the magnitudes in the paired readings arose by chance. Therefore, probability values exceeding five per cent indicate that the discrepancies in the paired values are not significant. It should be borne in mind that, in the statistical sense, the result 'not significant' is not so much a complete acceptance of the null hypothesis but rather an outcome of 'significance of difference not established'.

6.3.2 Coefficient of variation of errors

The coefficient of variation of errors (ω_j) was initially defined in Part VI of Bazant and Panula (1979) and subsequently used by Bazant et al., (1991a & b and 1992a, b & c), Bazant et al., (1993) and RILEM Model B3 (1995) to quantify the extent to which predicted creep values at different ages after loading deviate from the values measured at the relevant ages. The more accurate the prediction, the lower the value of ω_j.

The coefficient of variation of errors (ω_j) was used to quantify the extent to which predicted creep values at different ages after loading (determined by applying a particular model) deviate from the values measured at the relevant ages on the specimens of a particular concrete mix. This coefficient is expressed as a percentage and is defined by the following equations:

$$\omega_j = \{[\Sigma_i \Delta^2_{ij}/(n-1)]^{1/2}\}/J_i \tag{6.2}$$

in which

$$J_i = \Sigma_j J_{ij}/n \tag{6.3}$$

where,

ω_j	=	coefficient of variation for data set j
Δ_{ij}	=	the deviation (vertical) between the measured and predicted value for data point i on data set j
J_{ij}	=	the measured values (labelled by the subscript i) of specific creep in the data set number j
n	=	the total number of data points in the set

The overall coefficient of variation (ω_{all}) was used to estimate the average (pooled) coefficient of variation of a number of independent coefficients of variation, as follows:

$$\omega_{all} = [\Sigma_j \omega_j^2/N]^{1/2} \tag{6.4}$$

where,

N	=	the number sets considered

6.4 Discussion of Results

6.4.1 Elastic moduli of concrete

All the creep prediction models applied in this investigation include an empirical equation for the prediction of the elastic modulus of the concrete, which usually takes the 28 day compressive strength of the concrete into account (see Chapter 3). This equation is identical for the ACI 209 (1992) and AS 3600 (1988) methods (Pauw, 1990). In the case of the RILEM Model B3 (1995), the predicted elastic modulus is used in the calculation of the compliance function for additional creep due to drying ($C_d(t, \tau, t_o)$) and may be used to calculate the creep coefficient (ϕ) from the relevant compliance function, $C_o(t, \tau)$ or $C_d(t, \tau, t_o)$ from equations 3.59 or 3.67, respectively. However, in the case of all the other creep prediction models reviewed in Chapter 3, the predicted creep strain is directly dependent on the value of predicted elastic modulus. Therefore, the predicted elastic modulus is significant in the prediction of creep.

Figure 6.1 shows the elastic moduli predicted for each mix according to the different creep prediction methods and the average measured elastic moduli of the prisms of the six mixes (measured at 28 days after casting). The average of the measured values was taken as the average of all the values measured on the sealed and unsealed prisms of each mix.

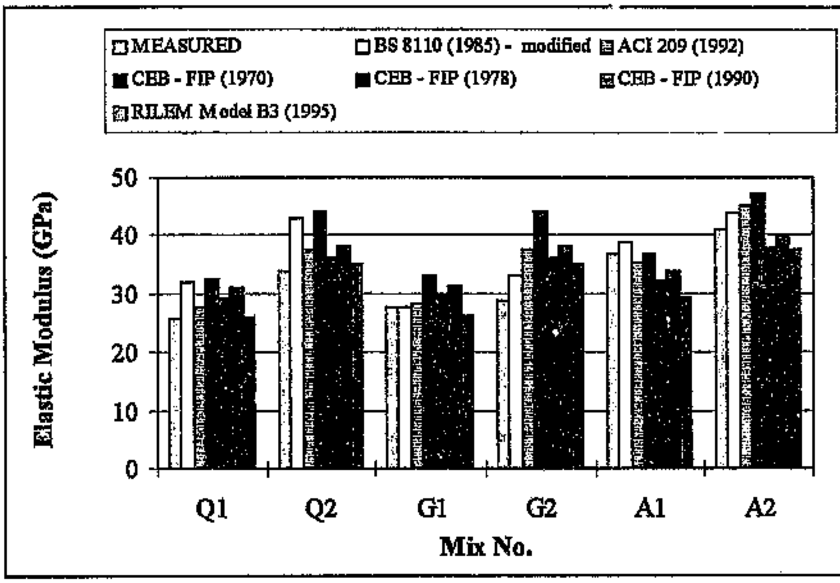


Figure 6.1 Measured and predicted elastic moduli values at 28 days after casting

The measured and predicted elastic moduli corresponding to the predicted values are given in Table 6.1. The statistical parameters at the bottom of the table pertain to the elastic moduli values of each mix (columns) and the t-Test results pertain to paired comparison of the measured values and the predicted values determined by each of the methods (rows).

It is evident from Table 6.1 that a relatively larger variance occurred in the values predicted for the lower w/c ratio mixes (Q2, G2 and A2) in comparison with the higher w/c ratio mixes. Furthermore, in the case of both w/c ratio mixes, the variance of the predicted elastic moduli with the use of different aggregates increases in the order quartzite, granite and andesite.

Table 6.1 Measured and predicted modulus of elasticity values for different concrete mixes

METHOD	Elastic Modulus of Concrete (GPa)						T-Test Results	
	Mix Q1	Mix Q2	Mix G1	Mix G2	Mix A1	Mix A2	Difference Significant ?	Level of Significance P (%)
MEASURED	25.8	34.0	27.8	28.9	36.7	40.9		
PREDICTED								
BS 8110 (1985) - modified	31.8	43.0	27.6	33.0	38.6	43.8	Yes	3.1
ACI 209 (1992) and AS 3600 (1988)	27.7	37.5	28.1	37.5	35.4	45.1	No	9.9
CEB - FIP (1970)	32.5	44.1	33.1	44.1	36.9	47.0	Yes	1.7
CEB - FIP (1978)	29.5	36.1	29.8	36.1	32.1	37.7	No	53.1
CEB - FIP (1990)	31.1	38.0	31.4	38.0	33.8	39.7	No	15.9
RILEM Model B3 (1995)	25.9	35.1	26.4	35.1	29.4	37.5	No	68.2
Average	29.8	39.0	29.4	37.3	34.3	41.8		
Standard Deviation	2.55	3.69	2.52	3.76	3.34	4.05		
Variance	6.53	13.63	6.37	14.17	11.14	16.40		

According to the t-Test results shown in Table 6.1, the discrepancies between the measured and predicted elastic moduli values, for the different mixes, were only significant in the case of the BS 8110 (1985) - modified method (P = 3.1) and the CEB-FIP (1970) method (P = 1.7). Figure 6.1 indicates the abovementioned significant differences to be attributable to the general over-estimation of elastic moduli values for each mix. The RILEM Model B3 (1995) yields the most accurate predictions (P = 68.2).

6.4.2 Rapid initial flow

As mentioned in Section 3.2.6, the CEB-FIP (1978) method enables the separate calculation of the reversible delayed elastic strain and irreversible flow components of the creep coefficient at any age t. The flow component is sub-divided into a component representing flow during the first 24 hours under load (rapid initial flow, $\beta_a(\tau)$) and a subsequent flow component.

The predicted and measured specific rapid initial flow creep values, for the six mixes included in the investigation, are given in Table 6.2. As the calculation of $\beta_a(\tau)$ does not consider whether moisture exchange between the concrete and the ambient environment is permitted, the measured value for each mix was taken as the average of the values measured for all the specimens (sealed and unsealed) of that mix.

Table 6.2 Measured and predicted specific rapid initial flow values

Mix	Specific Rapid Initial Flow (x10E-6/MPa)	
	Measured	Predicted
Q1	7.92	8.54
Q2	5.40	6.99
G1	10.41	8.49
G2	7.29	6.99
A1	6.03	7.92
A2	5.42	6.71
Difference Significant ?	No	
Level of Significance P (%)	40.8	

A t-Test conducted on the measured and predicted values in the above table indicated that the difference between the paired values is not significant (P = 40.8).

6.4.3 Total creep strain

The measured and predicted specific total creep values, for the concrete of each strength grade for each of the aggregate types tested, are included in Tables J.1 to J.6 in the appendix and shown in Figures 6.2 to 6.7. The statistics of the errors of each model in comparison with the test data are presented in Table 6.3.

Table 6.3 Coefficients of variation of errors (expressed as a percentage) of creep predictions for various models

Prediction Method	Coefficients of Variation (ω)						ω_{all}
	Mix Q1	Mix Q2	Mix G1	Mix G2	Mix A1	Mix A2	
BS 8110 (1985)	53.8	27.7	40.1	20.5	59.3	40.2	42.5
BS 8110 (1985) - MODIFIED	57.2	32.2	59.7	44.5	47.4	25.8	46.1
ACI 209 (1992)	52.6	36.3	45.7	45.1	60.8	58.4	50.5
AS 3600 (1988)	12.5		13.4		47.2		29.2
CEB - FIP (1970)	18.0	31.0	15.0	12.3	13.9	9.9	18.0
CEB - FIP (1978)	66.0	148.6	53.9	95.1	65.6	112.8	96.1
CEB - FIP (1990)	32.7	19.8	27.7	31.2	39.6	38.3	32.2
RILEM Model B3 (1995)	45.5	29.2	33.0	21.9	45.3	32.6	35.6
ω_{all}	46.0	62.6	39.4	46.4	49.8	54.8	

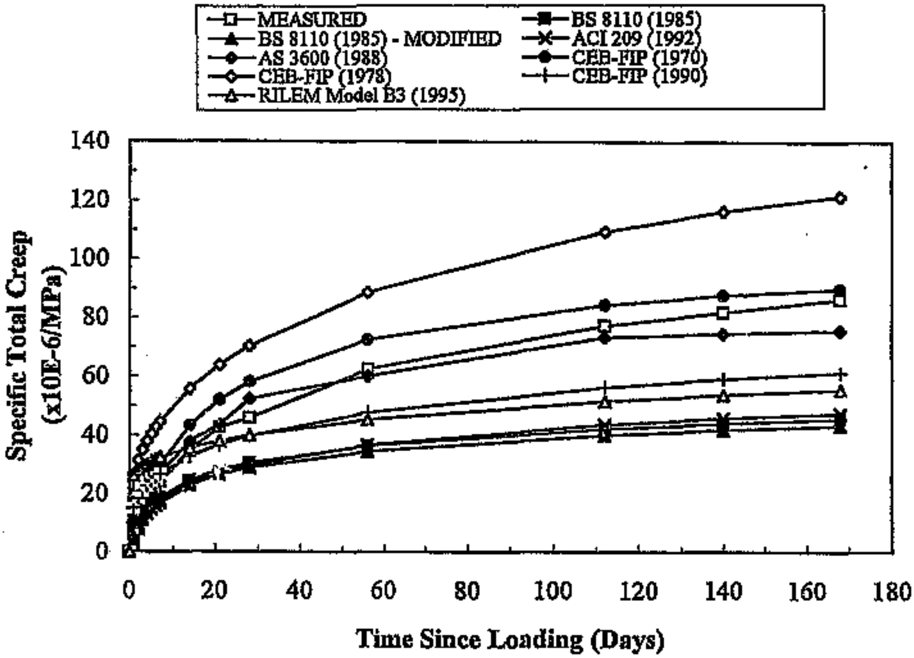


Figure 6.2 Measured and predicted specific total creep versus time since loading for mix Q1 specimens

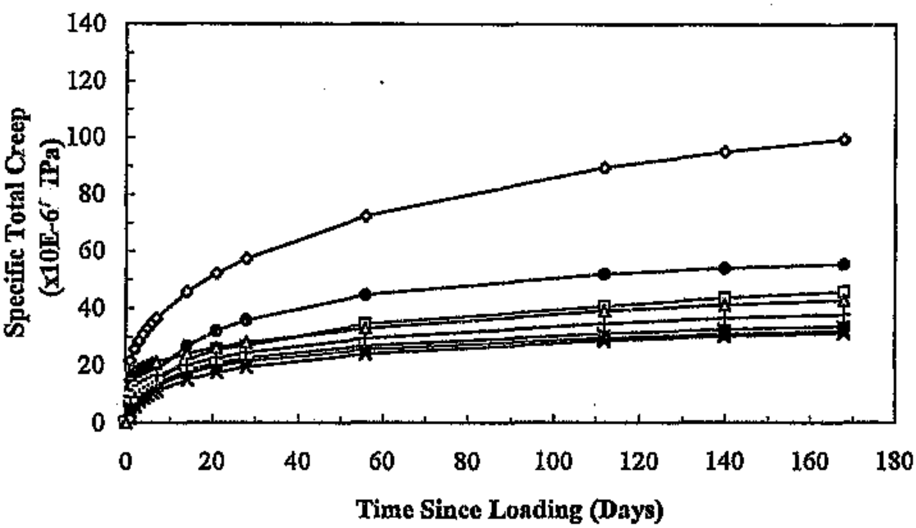


Figure 6.3 Measured and predicted specific total creep versus time since loading for mix Q2 specimens

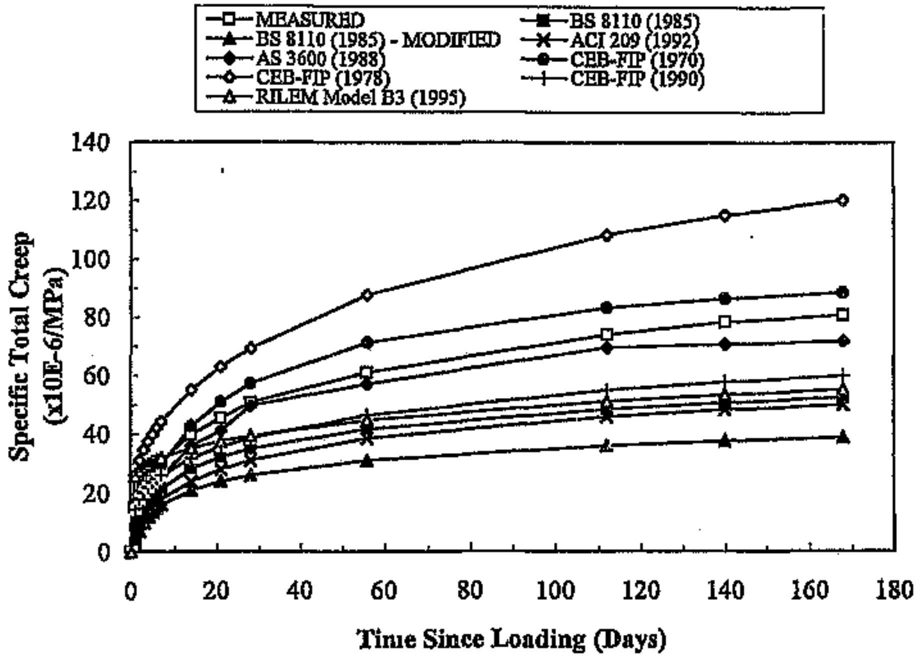


Figure 6.4 Measured and predicted specific total creep versus time since loading for mix G1 specimens

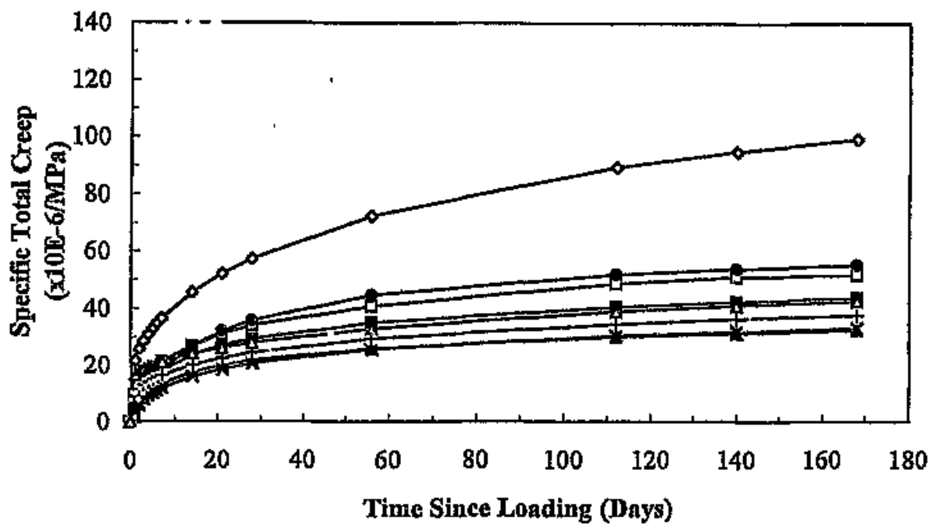


Figure 6.5 Measured and predicted specific total creep versus time since loading for mix G2 specimens

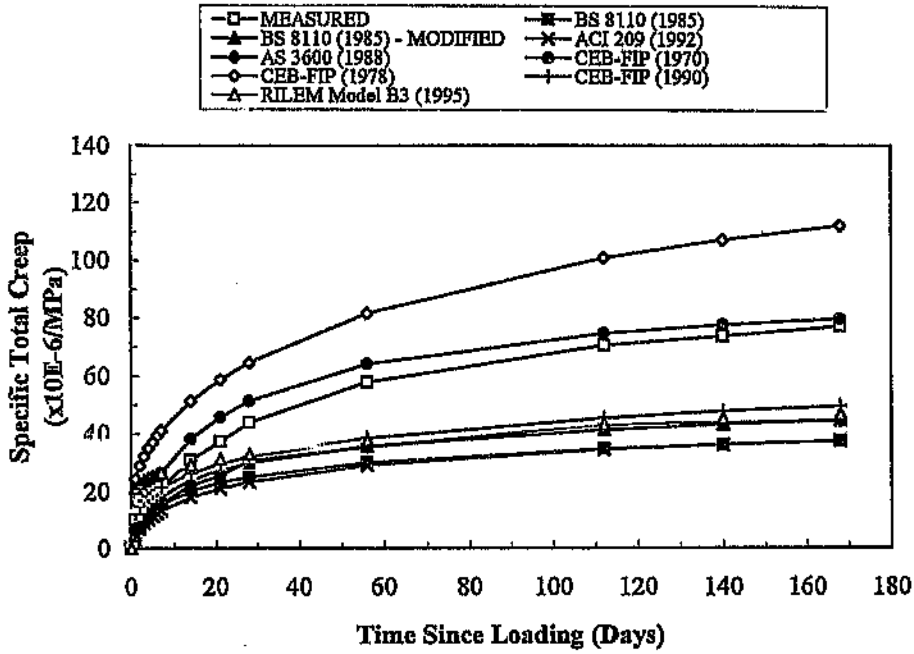


Figure 6.6 Measured and predicted specific total creep versus time since loading for mix A1 specimens

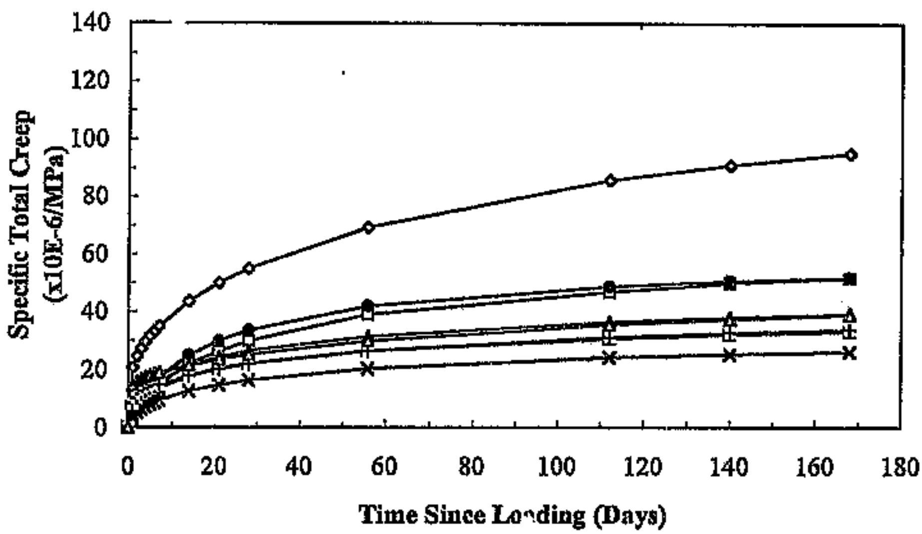


Figure 6.7 Measured and predicted specific total creep versus time since loading for mix A2 specimens

From Figures 6.2 to 6.7 it is clear that for all six mixes, for the loading period considered, the CEB-FIP (1978) method overpredicted the specific total creep yielding an overall coefficient of variation (ω_{all}) of 96.1. The average overprediction yielded by the CEB-FIP (1978) method for each of the mixes lies within the range of 49 to 155 per cent higher than the measured values. Furthermore, the degree of over prediction increased with time since loading.

The CEB-FIP (1970) method generally underpredicted the specific total creep at earlier ages (within the first eight days after loading) and then yielded overpredictions, the extent of which decreased with time, for the remainder of the period considered. For the six mixes, the overprediction at 168 days after loading ranged from 0.3 to 21 per cent. The remaining creep prediction methods generally underpredicted the specific total creep.

A comparison of all the underpredicted values revealed the following:

- In the case of the both the Q1 and G1 mixes the most and least accurate predictions were yielded by the AS 3600 (1988) method and BS 8110 (1985) - Modified method, respectively. However this trend is not applicable in the case of the other mix with w/c ratio of 0.56 (i.e. mix A1) where the CEB-FIP (1990) method yielded the most accurate and the ACI 209 (1992) the least accurate predictions.

- For the mixes with the w/c ratio of 0.4 (mixes Q2, G2 and A2) no particular method yielded the most accurate predictions. However, the ACI 209 (1992) yielded the least accurate prediction in all three of these mixes.

- The accuracy of the predictions at any time after loading was greater in the case of the concrete with the lower w/c ratio mix of each aggregate type, in comparison with the higher w/c ratio mix of that aggregate type.

On the basis of all the predictions the following was concluded:

Only in the case of the andesite concretes did one method, the CEB-FIP (1970), yield the most accurate predictions for both w/c ratio mixes.

The methods that yielded the most accurate prediction, greatest overprediction and greatest under prediction (on average) were the CEB-FIP, 1970 (ω_{all} = 18), CEB-FIP, 1978 (ω_{all} = 96.1) and the ACI 209, 1992 (ω_{all} = 50.5), respectively.

Referring to the discussion on the predicted elastic moduli values (Section 6.4.1), if the CEB-FIP (1970) did not overpredict the elastic modulus for all six mixes, the extent of overprediction of specific total creep would be larger and hence this method would be less accurate. Therefore, assuming that the elastic moduli values predicted by the different methods were accurate, the creep coefficients yielded by the CEB-FIP (1978) would have to be lower in order to result in more accurate predicted specific creep values. Similarly, the creep coefficients of all the methods that underestimated specific creep (excluding the RILEM Model B3, 1995) would need to be higher to improve the accuracy of these methods.

The above results also indicate that the accuracy of the predictions did not increase with the complexity of the method used. In particular, this is evident in the case of the CEB-FIP (1978) and CEB-FIP (1970) methods, where the former method is a relatively complex method and yielded the least accurate overall results, whereas the latter method yielded the most accurate overall results.

An increase in the number of variables accounted for by a prediction method (see Table 3.1) does not necessarily increase the accuracy of the predicted values. This was observed in the case of the ACI 209 (1992) method.

6.4.4 Basic creep strain

The measured and predicted specific basic creep strains, for the concrete of each strength grade for each of the aggregate types investigated, are included in Tables J.1 to J.6 in Appendix J and shown in Figures 6.8 to 6.13. The statistics of the errors of each model in comparison with the test data are presented in Table 6.4, which is located after the abovementioned figures.

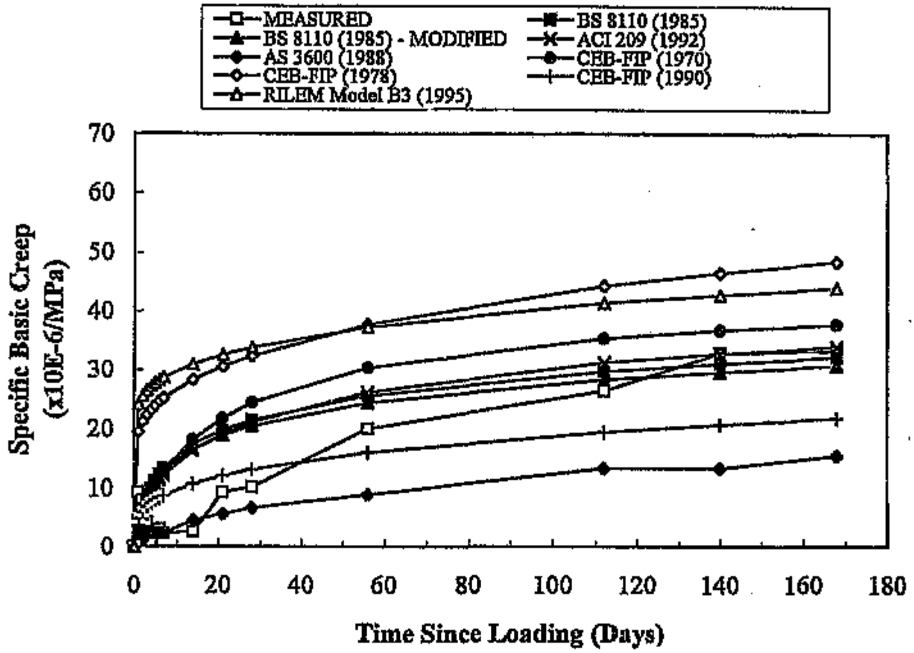


Figure 6.8 Measured and predicted specific basic creep versus time since loading for mix Q1 specimens

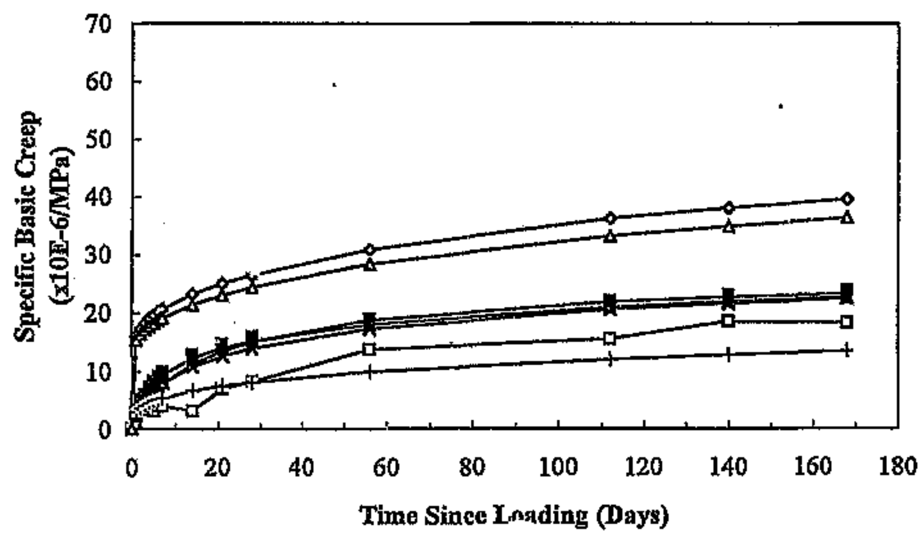


Figure 6.9 Measured and predicted specific basic creep versus time since loading for mix Q2 specimens

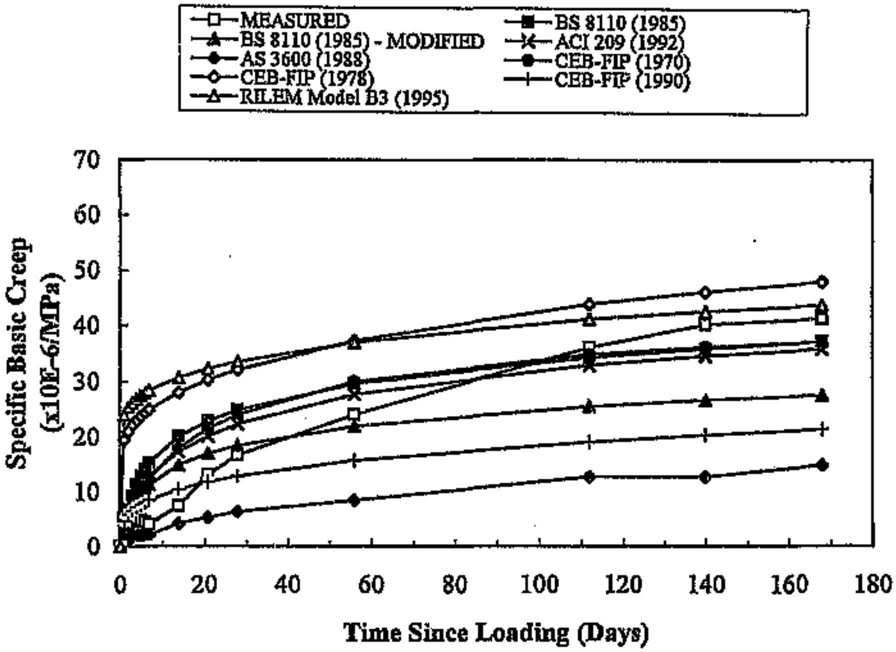


Figure 6.10 Measured and predicted specific basic creep versus time since loading for mix G1 specimens

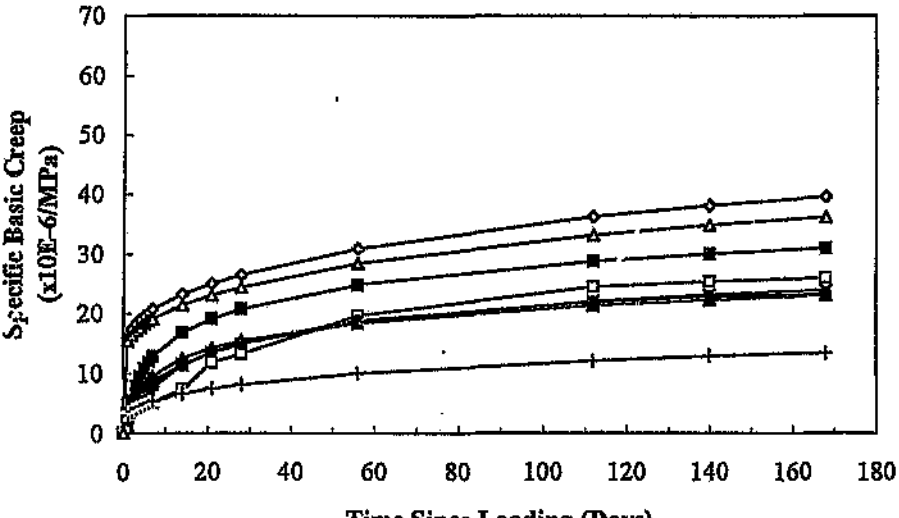


Figure 6.11 Measured and predicted specific basic creep versus time since loading for mix G2 specimens

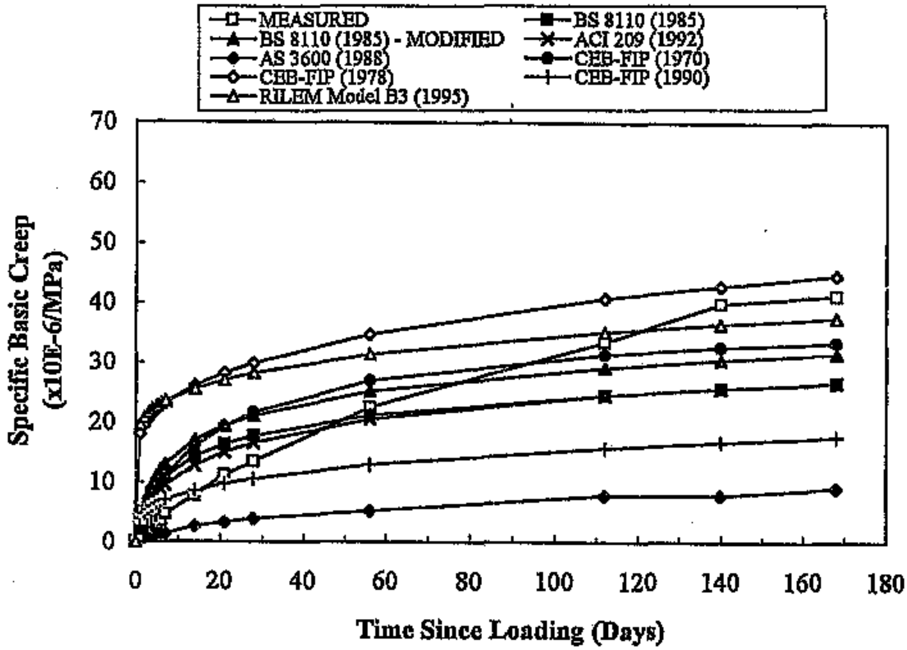


Figure 6.12 Measured and predicted specific basic creep versus time since loading for mix A1 specimens

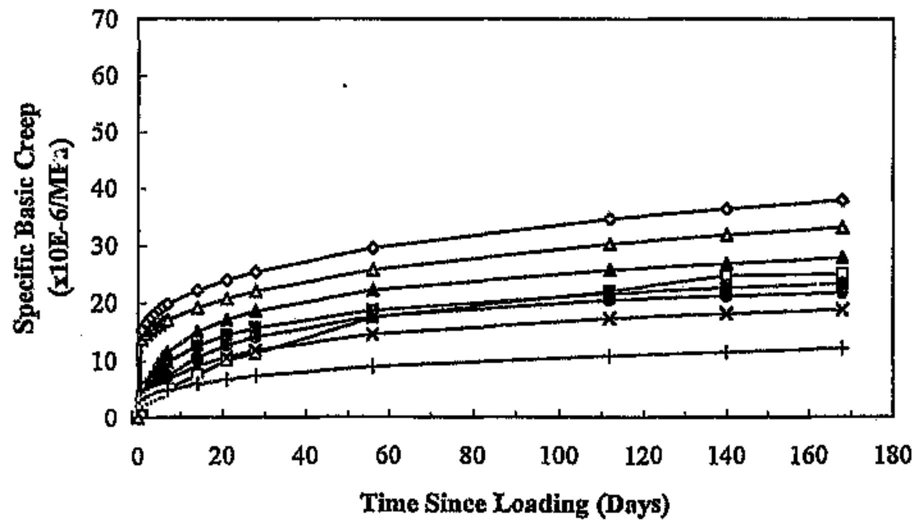


Figure 6.13 Measured and predicted specific basic creep versus time since loading for mix A2 specimens

Table 6.4 Coefficients of variation of errors (expressed as a percentage) of basic creep predictions for various models

Prediction Method	Coefficients of Variation (ω)						ω_{all}
	Mix Q1	Mix Q2	Mix G1	Mix G2	Mix A1	Mix A2	
BS 8110 (1985)	72.9	72.3	49.2	48.8	54.4	30.3	56.6
BS 8110 (1985) - MODIFIED	68.5	64.9	48.2	25.0	52.2	46.9	52.9
ACI 209 (1992)	67.2	52.9	39.0	18.3	51.1	28.9	45.9
AS 3600 (1988)	79.8		91.1		114.0		96.0
CEB - FIP (1970)	81.4	60.1	39.2	19.0	44.3	21.1	49.2
CEB - FIP (1978)	171.1	218.1	107.4	115.1	114.8	127.3	147.8
CEB - FIP (1990)	57.8	35.9	63.9	58.0	78.8	62.0	60.7
RILEM Model B3 (1995)	188.3	193.2	124.4	98.9	114.7	98.2	141.9
ω_{all}	105.3	120.7	76.8	65.6	83.5	69.8	

The following is evident from Figures 6.8 to 6.13 and Table 6.4.

- The predictions are more accurate for the lower w/c ratio mix of each aggregate type. This is as a result of the relatively lower measured specific basic creep rate in the case of the lower w/c mix of each aggregate type.

- The CEB-FIP (1990) and AS 3600 (1988) methods generally underpredicted the specific basic creep values at all ages. Note that the AS 3600 (1988) method was not applied to predict values for the lower w/c ratio mixes (0.4).

- The CEB-FIP (1978) method overpredicted the specific basic creep for all six mixes and was the least accurate method, yielding the highest overall coefficient of variation (ω_{all} = 147.8). This method also yielded the lowest overall coefficient of variation for the specific total creep predictions.

- When considering the values predicted by the different methods for all six mixes, the ACI 209 (1992) method yielded the best overall predictions (ω_{all} = 45.9).

- The RILEM Model B3 (1995) overpredicted the specific basic creep at all ages in five of the six mixes (mix A1 excluded).

- In the case of five of the mixes (mix Q2 excluded), the application of the ACI 209 (1992) method resulted in the overprediction of the specific basic creep at earlier ages and underprediction at later ages. This trend was also applicable to predictions made by the BS 8110 (1985), BS 8110 (1985) - modified and the CEB-FIP (1970) methods for a minimum of four mixes per method.

- The lowest coefficient of variation for both w/c ratio mixes of each aggregate type for the quartzite, granite and andesite was yielded by the CEB-FIP (1990), ACI (1992) and CEB-FIP (1970) methods, respectively. These results indicate that one particular method is best suited for predicting the specific basic creep of concrete containing quartzite or granite or andesite. However, this does not imply that all the prediction methods are generally more accurate for concrete containing a particular aggregate type.

- With the exception of the specific basic creep versus time curve corresponding to the values predicted by the BS 8110 (1985) and ACI 209 (1992) methods, the relative positions of the other curves are generally identical for all six mixes. These positions do not correspond with those of the same methods for the specific total creep versus time since loading relationships (see Figures 6.2 to 6.7).

In conclusion, a comparison of the measured versus predicted results for the two w/c ratios of each aggregate type revealed that no particular trend was attributable to the aggregate stiffness or type.

As was the case in the specific total creep predictions, the accuracy of the specific basic creep predictions did not increase with the complexity of the method used. This is evident in the overall coefficients of variation yielded by the CEB-FIP (1978) and the RILEM Model B3 (1995) methods which were much higher than the overall coefficient of variation yielded by the relatively simple CEB-FIP (1970) method.

6.5 Comparison of Results Obtained with Results of Other Investigations

6.5.1 Davis and Alexander (1992)

The BS 8110 (1985) creep prediction method does not make it possible to predict structural deformations with much precision (Davis and Alexander, 1992). This is confirmed by the results of this investigation where neither the SABS 8110 (1985) method nor the SABS 8110 (1985) Modified method yielded the most accurate predicted specific total creep or specific basic creep for any of the six mixes. Only in the case of the specific total creep predictions for the andesite concretes (mixes A1 and A2) did the SABS 8110 (1985) Modified method yield more accurate results than those predicted by the SABS 8110 (1985).

With regard to the specific basic creep values, the BS 8110 (1985) Modified method yielded a lower coefficient of variation than the BS 8110 (1985) method for all the mixes except mix A2.

As the measured specific total creep values were calculated (using equation 5.3) from the strains recorded on the test specimens, their degree of accuracy was assumed to be greater than the "measured " specific basic creep values which were affected by the inferred autogenous shrinkage strain values (see equation 5.4). Revised relative creep coefficients (to be applied with the BS 8110 (1985) method) were calculated for each of the aggregate types included in this investigation from the measured and predicted specific total creep values for each of the mixes. The revised coefficients were determined for each w/c ratio of each aggregate type (i.e each mix) by taking the average of the factors by which the predicted values at the various ages of that mix must be multiplied in order to obtain the measured value. These coefficients are given together with the corresponding coefficients determined by Davis and Alexander (1992) in Table 6.5.

Table 6.5 Relative creep coefficients for different aggregate types

Aggregate Type	Relative Creep Coefficients			
	According to Davis and Alexander (1992)	Revised		
		w/c = 0.56	w/c = 0.40	Average for both w/c ratios
Quartzite (Ferro)	0.95	1.61	1.36	1.49
Granite (Jukskei)	0.74	1.86	1.48	1.67
Andesite (Eikenhof)	1.19	1.99	1.55	1.77

The relative creep coefficients established by Davis and Alexander (1992) indicate that the creep values predicted by the BS 8110 (1985) method for concrete containing quartzite (Ferro) or granite (Jukskei) aggregates should be reduced. The results of this investigation indicate that the values predicted by the BS 8110 (1985) method should be increased for these aggregates.

The magnitude of the average revised relative creep coefficients (in Table 6.5) indicates that the general inaccuracy of the BS 8110 (1985) method in comparison with some of the other methods is probably due to more than the stiffness of the aggregate type. This method does not account for a number of secondary variables such as temperature, curing conditions, cement content and w/c ratio (Davis and Alexander, 1992).

On the basis of the results in the above table and in view of the fact that only three aggregates were included in this investigation, further research is justified to either re-define the relative creep coefficients which were introduced by Davis and Alexander (1992) or confirm those given in Table 6.5. However, such research may not indicate an improved accuracy of the BS 8110 method, in comparison to other existing methods, in situations where the intrinsic and/or extrinsic variables differ significantly from those pertaining to the tests conducted in this project.

It is therefore recommended that the BS 8110 (1985) method which was incorporated in the SABS 0100 (1992) be replaced by one of the other existing methods or by a newly developed model. The former option would be preferable.

6.5.2 RILEM Data Bank

In the justification of the RILEM Model B3 (1995), comparisons were made between the coefficients of variation of errors (ω_j) for specific creep at drying (total) and specific basic creep predictions for the RILEM Model B3 (1995), the ACI 209 (1992) and the CEB-FIP (1990) methods (RILEM Model B3, 1995). The data used in these comparisons derived from the RILEM Data Bank, which was compiled by subcommittee 5 of RILEM Committee TC-107 (1995), comprising approximately 15 000 data points from various laboratories around the world. The drying (total) creep and basic creep comparisons are discussed below.

Total creep

The coefficients of variation for the specific total creep comparisons are given in Table 6.6.

Table 6.6 Coefficients of variation for specific total creep predictions for the RILEM Model B3 (1995), ACI 209 (1992) and CEB-FIP (1990) methods (after RILEM Model B3, 1995)

Test Data	Coefficients of Variation (ω)		
	RILEM Model B3 (1995)	ACI 209 (1992)	CEB-FIP (1990)
Hansen and Mattock (1966)	5.8	32.1	11.9
Keeton (1965)	31.4	46.3	37.9
Troxell et al., (1958)	5.9	33.0	7.9
L'Hermite et al., (1965)	14.0	55.8	25.5
Rostasy et al., (1972)	6.5	20.9	14.8
York et al., (1970)	5.8	42.1	45.1
McDonald (1975)	10.9	40.4	38.9
Hummel et al., (1962)	15.3	46.2	24.6
L'Hermite and Mamillan (1970)	20.6	62.5	15.2
Mossiossian and Gamble (1972)	11.3	71.7	30.8
Maity and Meyers (1970)	62.8	45.9	83.7
Russel and Burg (1993)	10.7	41.2	19.1
ω_{all}	23.1	46.8	35.5

From Table 6.6 it is evident that the overall coefficients of variation for the three models increase in the order Model B3 (1995) (ω_{all} = 23.1), CEB-FIP (1990) (ω_{all} = 35.5), and ACI 209 (1992) (ω_{all} = 46.8). The total creep results for the present investigation (Table 6.3) are in disagreement with the above relative order in that the CEB-FIP (1990) method yielded a lower value (ω_{all} = 32.2) than the Model B3 (ω_{all} = 35.6). This disagreement may be attributable to the fact that the RILEM data bank does not include data for South African concretes. The relative magnitude of the overall coefficients of variation, with the three prediction methods, established in this investigation is in agreement with that obtained by York et al., (1970) and Maity and Meyers (1970) in Table 6.6.

However, the results of this investigation agree with the findings presented in Table 6.6 in the following two aspects:

- the values predicted by the ACI 209 (1992) method in this investigation also yielded a relatively higher overall coefficient of variation (ω_{all} = 50.5) in comparison with the other two methods and,

- the overall coefficient of variation for the CEB-FIP (1990) and ACI 209 (1992) are of a similar order of magnitude in the two sets of results compared.

Basic creep

The coefficients of variation for the specific basic creep comparisons are given in Table 6.7. It is evident from this table that the overall coefficients of variation pertaining to basic creep for the three models increase in the same order as the overall coefficients of variation for drying creep (i.e. Model B3 (1995), CEB-FIP (1990) and ACI 209 (1992)). The basic creep results of this investigation (Table 6.4) indicate an opposite trend to that described above, with the overall coefficients of variation of the three relevant methods increasing in the order ACI 209 (1992), CEB-FIP (1990) and Model B3 (1995). In addition, a large discrepancy exists between the magnitude of the overall coefficient of variation obtained by each method in this investigation and that obtained by the corresponding method on the basis of the RILEM data bank.

Table 6.7 Coefficients of variation for specific basic creep predictions for the RILEM Model B3 (1995), ACI 209 (1992) and CEB-FIP (1990) methods (after RILEM Model B3, 1995)

Test Data	Coefficients of Variation (ω)		
	RILEM Model B3 (1995)	ACI 209 (1992)	CEB-FIP (1990)
Keeton (1965)	19.0	37.5	42.8
Kommendant et al., (1976)	15.3	31.8	8.1
L'Hermite et al., (1965)	49.4	133.4	66.2
Rostasy et al., (1972)	15.2	47.6	5.0
Troxell et al., (1958)	4.6	13.9	6.2
York et al., (1970)	5.6	37.7	12.8
McDonald (1975)	6.9	48.4	22.2
Maity and Meyers (1970)	33.8	30.0	15.7
Mossiossian and Gamble (1972)	18.6	51.5	47.3
Harboe et al., (1958) (Ross Dam)	14.1	51.2	31.1
Browne and Bamforth (1975) (Wylfa vessel)	44.7	47.3	53.3
Harboe et al., (1958) (Shasta Dam)	22.7	107.8	43.1
Brooks and Wainwright (1983)	12.6	14.9	15.4
Pirtz (1968) (Dworshak Dam)	12.5	58.2	32.5
Harboe et al., (1958) (Canyon ferry Dam)	33.3	70.2	56.9
Russel and Burg (1993) (Water Tower Place)	15.7	19.3	31.5
Hanson (1953)	14.1	63.3	12.1
ω_{all}	23.6	58.1	35.0

In view of the above, the basic creep results obtained in this investigation appear to be questionable. The probable inaccuracy of the basic creep results may be attributable to the inferred autogenous shrinkage values.

6.5.3 Brooks et al., (1992)

Brooks et al., (1992) carried out laboratory creep tests on two high strength concrete prisms (seven day cube strength of 50 MPa). The measured total creep at eight months (after casting) was compared to the creep predicted at the same age using a number of methods including the CEB-FIP (1970), CEB-FIP (1978) and BS 8110 (1985) methods.

The ratios of the predicted to the measured strains in the case of the CEB-FIP (1970), CEB-FIP (1978) and BS 8110 (1985) were 2.35, 1.52 and 1.6, respectively. Therefore, all the methods overestimated the total creep strain. The only trend that is in agreement with the results of this investigation is that both the CEB-FIP (1970) and the CEB-FIP (1978) methods overestimated the creep. Nevertheless, there is no cause for concern, over the differences in results of this research and Brooks' research, as the results of the latter project are based on only two samples of high strength concrete and many of the intrinsic and/or extrinsic factors differed from those of the this project.

6.5.4 McDonald et al., (1988)

McDonald et al. (1988) assessed the accuracy of a number of creep prediction methods including the CEB-FIP (1978) and AS 3600 (1988) methods. Only total creep was considered. The experimental data, which was compared to the values predicted at the corresponding ages by the different models, comprised over 1000 data points from 29 creep tests (conducted on Australian concretes) from five different sources.

The results of this assessment led to the following conclusions:

- None of the prediction methods investigated exhibited a consistent trend with regards to over and/or underprediction.

- The overall coefficients of variation (ω_{all}) yielded by the CEB-FIP (1978) and AS 3600 (1988) methods were 72.1 and 22.4 per cent, respectively. These overall coefficients of variation (ω_{all}) are in fair agreement with the coefficients established for the corresponding methods in this investigation (see Table 6.3).

6.5.5 Alexander (1986)

Alexander (1986) gives details of an investigation carried out to assess the accuracy of the C & CA (1979), CEB-FIP (1970), CEB-FIP (1978) and ACI (1982) creep prediction methods when applied to South African concretes. The C & CA (1979) and ACI (1982) methods appear to be identical to the BS 8110 (1985) and ACI 209 (1992) methods, respectively. With regards to the C & CA (1979) method, the procedures described by Alexander (1986) and in this investigation differed in that equations 3.2 and 3.3 were used to calculate the elastic modulus in the former case as opposed to the application of equation 3.4 in the latter case.

The intrinsic and/or extrinsic parameters pertaining to Alexanders' experimental specimens are as follows:

- the strength range varied from 1.7 to 56.5 MPa at loading
- OPC was used
- loading ages ranged from 14 to 74 days
- concrete prisms (102 x 102 x 200 mm) were cured in water at a temperature of 20 °C until loading
- the ambient relative humidity was maintained at 50 ± 10 per cent
- all the measured creep strains were normalised for a paste content of 35 per cent

The measured and predicted creep strains at 60 days for different stress : strength ratios are shown in Figure 6.14.

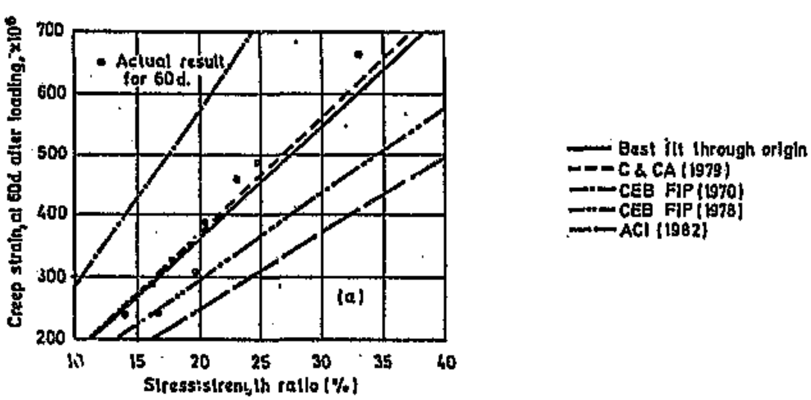


Figure 6.14 Comparison between measured and predicted creep strains of South African concretes at 60 days after loading (after Alexander, 1986)

A comparison of the results in Figure 6.14 to those obtained for total creep at 60 days after loading in this investigation (Figures 6.2 to 6.7) led to the conclusion that the only similarity in the comparison is that the ACI (1992/1982) method and the CEB-FIP (1970) methods underestimated and overestimated the creep strain, respectively. Furthermore, the results of this investigation show the extent of overprediction yielded by the CEB-FIP (1970) to be significantly less than that established by Alexander (1986).

In conclusion, the results of this investigation generally disagree with those obtained by Alexander (1986).

6.6 Conclusions

6.6.1 Elastic moduli of concrete

All the creep prediction methods included in this project consider the value of a predicted elastic modulus of the concrete in calculating predicted creep strain. A comparison of the predicted elastic moduli, determined for each mix by the different creep prediction methods, with the measured elastic moduli of the relevant mixes indicated that the differences were only significant in the case of the BS 8110 (1985) modified method and the CEB-FIP (1970) ethod.

6.6.2 Total and basic creep

A comparison of the experimental specific total creep and specific basic creep values from this investigation with those predicted at the corresponding ages by the BS 8110 (1985), BS 8110 (1985) - Modified, ACI 209 (1992), AS 3600 (1988), CEB-FIP (1970), CEB-FIP (1978), CEB-FIP (1990) and the RILEM Model B3 (1995) methods revealed the following:

Total creep

- The CEB-FIP (1978) method overpredicted the specific total creep for all the mixes;

- The CEB-FIP (1970) method generally underpredicted the specific total creep at earlier ages (within the first eight days after loading) and then yielded overpredictions;

- The remaining creep prediction methods generally underpredicted the specific total creep;

- The accuracy of the predictions at any time since loading was generally greater in the case of the concrete with the lower w/c ratio mix of each aggregate type, in comparison with the higher w/c ratio mix of that aggregate type;

- Only in the case of the andesite concretes did one method, the CEB-FIP (1970), yield the most accurate predictions for both w/c ratio mixes.

- The methods that yielded the most accurate prediction, greatest overprediction and greatest under prediction (on average) were the CEB-FIP (1970), CEB-FIP (1978) and the ACI 209 (1992), respectively.

Basic creep

- The CEB-FIP (1978) and RILEM Model B3 (1995) methods generally significantly overpredicted the specific basic creep for all the mixes;

- The CEB-FIP (1990) and AS 3600 (1988) methods generally underpredicted the specific basic creep values at all ages, with the latter being the more accurate of the two methods;

- The remaining methods generally overpredicted the specific basic creep at earlier ages and underpredicted this value at later ages;

- The predictions are more accurate for the lower w/c ratio mix of each aggregate type;

- The methods that yielded the most accurate and least accurate predictions (on average) were the ACI 209 (1992) and the CEB-FIP (1978) methods, respectively.

All the above creep results indicate that the accuracy of the predictions does not increase with the complexity of the method applied or with increasing number of variables accounted for in the method. Furthermore, none of the observed trends were attributable to the aggregate stiffness or type.

6.6.3 Comparison of results from this project with results of other investigations

A comparison of the accuracy of the prediction methods included in this investigation with the accuracy determined for some of these methods in separate research projects by Davis and Alexander (1992), RILEM Data Bank, McDonald et al., (1988) and Alexander (1986) revealed the following.

Davis and Alexander (1992)

The relative creep coefficients established by Davis and Alexander (1992) indicate that the creep values predicted by the BS 8110 (1985) method for concrete containing quartzite (Ferro) or granite (Jukskei) aggregates should be reduced. The results of this investigation indicate that the values predicted by the BS 8110 (1985) method should be increased for these aggregates.

The magnitude of the average revised relative creep coefficients indicates that the general inaccuracy of the BS 8110 (1985) method in comparison with some of the other methods is probably due to more than the stiffness of the aggregate type.

RILEM Data Bank

Results from the RILEM Data Bank, comparing the relative accuracy of the total creep and basic creep predictions yielded by the RILEM Model B3 (1995), CEB-FIP (1990) and ACI 209 (1992) methods showed the accuracy of these methods to increase in the abovementioned order. The total creep results of this investigation are in disagreement with the above relative order in that the CEB-FIP (1990) method yielded a more accurate results than the RILEM Model B3 (1995). This disagreement may be attributable to the fact that the RILEM data bank does not include data for South African concretes. However, the agreements in the two sets of results compared outweigh the above disagreement. In the case of the accuracy of the basic creep predictions, significant disagreement exists between the results of this investigation and those based on the RILEM Data Bank. Therefore, the basic creep results obtained in this investigation appear to be questionable. The probable inaccuracy of the basic creep results may be attributable to the magnitudes of the inferred autogenous shrinkage values.

Brooks et al., (1992)

Brooks et al., (1992) compared the accuracy of total creep predictions using a number of methods including the CEB-FIP (1970), CEB-FIP (1978) and BS 8110 (1985) methods and concluded that the all abovementioned methods overestimated the creep strain. This is in agreement with the predictions yielded by the CEB-FIP (1970) and CEB-FIP (1978) methods in this investigation. However, differences exist in the degree of over estimation yielded by these methods in the two sets of data compared. Nevertheless, there is no cause for concern, over the differences in results of this research and Brooks' research, as the results of the latter project are based on only two samples of high strength concrete and many of the intrinsic and/or extrinsic factors differed from those of the this project.

McDonald et al., (1988)

McDonald et al. (1988) assessed the accuracy of total creep predictions yielded by a number of creep prediction methods including the CEB-FIP (1978) and AS 3600 (1988) methods, for Australian concretes. The results of this assessment concluded that the AS 3600 (1988) method was more accurate than the CEB-FIP (1978) method. This is in agreement with the results of this project.

Alexander (1986)

Alexander (1986) gives details of an investigation carried out to assess the accuracy of the BS 8110 (1985), CEB-FIP (1970), CEB-FIP (1978) and ACI (1982/1992) creep prediction methods when applied to South African concretes.

A comparison of the results of Alexander (1986) with those obtained for total creep at 60 days after loading in this investigation led to the conclusion that the only similarity in the comparison is that the ACI (1992/1982) method and the

CEB-FIP (1970) methods underestimated and overestimated the creep strain, respectively. Furthermore, the results of this investigation show the extent of overprediction yielded by the CEB-FIP (1970) to be significantly less than that established by Alexander (1986). The results of this investigation generally disagree with those obtained by Alexander (1986).

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

7.1.1 Measured creep strains

Specific creep (creep strain per unit applied stress) values were used for comparing the creep strains exhibited by the different concrete mixes included in this investigation. The results of this research revealed the following:

Effect of different w/c ratios on creep strain

For the concretes made with each of the aggregate types, at any age after loading, the mix with the lower w/c ratio (0.4) yielded a lower specific creep value. This trend was observed in both the total and basic creep results and is agreement with the findings of Reutz (1965), Ballim (1983), Smadi et al., (1987), Addis (1992) and Fiorato (1995).

Effect of aggregate stiffness on creep strain

The average elastic modulus values of the quartzite, granite and andesite were determined as 73, 70 and 89 GPa, respectively.

The creep test results indicated that, in the case of both total and basic creep, the specific creep of concrete with a particular w/c ratio (for most of the test period) with the use of different aggregates, generally increased in the order quartzite, andesite and granite. This indicates that the creep behaviour of concrete containing aggregate with a relatively high elastic modulus is significantly influenced by some other property of the aggregate, rather than stiffness.

The specific total creep values at six months after loading were adjusted to eliminate the influence of different w/c ratios on creep. These results indicated that a significant positive correlation exists between the specific total creep of concrete and the elastic modulus of the aggregate used in the concrete. For the concretes of each aggregate type, the higher the elastic modulus of the aggregate, the more the relative creep of the concrete. An identical trend was established using data from Davis and Alexander (1992) for the same aggregates as those considered in this investigation but pertaining to an age of five years after loading. These correlations show an opposite trend to those established by Rusch et al., (1962) and The Concrete Society (1974), which indicate that the higher the elastic modulus of the aggregate, the greater the restraint offered by the aggregate to the creep of the paste. However, these relationships indicate creep of concrete to be relatively insensitive to aggregate stiffness in the case of aggregates with a modulus of elasticity in excess of approximately 70 GPa (Rusch et al., 1962 and The Concrete Society, 1974).

The unexpected abovementioned results appear to be attributable to the stress strain behaviour of the aggregate/paste interfacial zone. Consideration of the behaviour of the interfacial zone, which is dependent on bond strength and density of this zone, is of significance and should not be overlooked by assuming that concrete is a two phase material (Nilsen and Monteiro, (1993), Alexander, (1991), Alexander and Davis (1992), Alexander (1993a) and Mindess and Alexander (1995)). Therefore, in this investigation the measured creep that was assumed to have taken place in the paste may have included the magnitude of strains which occurred at the interfacial zone.

Comparison of results with those of Davis and Alexander (1992)

The relative magnitudes of concrete creep with the use of different aggregates, obtained for the mixes in this investigation, are in disagreement with the results of research conducted by Davis and Alexander (1992) on creep of concretes with various aggregates, including those used in this investigation. Their results show concrete creep with the use of these aggregates to increase in the order granite, quartzite and andesite. These disagreements appear to be due to the analytical approach of the work of Davis and Alexander (1992) and to a lesser extent due to variations in the properties of aggregates from the same source.

Accuracy of autogenous shrinkage strains

The calculated specific basic creep values for each of the mixes included in the investigation, for different ages of loading, included a consideration of the autogenous shrinkage strain values. As no provision was made for the measurement of autogenous shrinkage strains, these strains were inferred from the results of research conducted by Alexander (1994b). The data used for inferring the autogenous strains were unusually high, comprising strains of magnitudes approximately three times higher than anticipated at an age of 360 days (Alexander, 1998). In addition, this data pertained to concrete containing only quartzite aggregates. Therefore, the accuracy of the specific basic creep values in this investigation has not been verified.

7.1.2 Comparison of measured and predicted creep strains

Accuracy of predictions

A comparison of the experimental basic creep and total creep values from this investigation with those predicted at the corresponding ages for the loading period considered by the BS 8110 (1985), BS 8110 (1985) Modified, ACI 209 (1992), AS 3600 (1988), CEB-FIP (1970), CEB-FIP (1978), CEB-FIP (1990) and the RILEM Model B3 (1995) methods revealed the following:

The creep strains predicted by the different methods vary widely. The specific total and basic creep values predicted by the different methods for the loading period considered (six months) generally indicated the trends that are shown in Table 7.1, with regards to overpredictions and underpredictions.

It was established that, for all the mixes, the relative positions of the creep versus time curves for the specific total creep did not correspond with those of the same methods for the specific basic creep.

In the case of specific total creep, the methods that yielded the most and least accurate predictions (on average) were the CEB-FIP (1970) and CEB-FIP (1978), respectively. However, the ACI 209 (1992) and CEB-FIP (1978) methods yielded the most and least accurate specific basic predictions, respectively.

The following conclusions were established for both the total and basic specific creep predictions:

- The CEB-FIP (1978) method significantly overestimated the specific creep strain values and was the least accurate method;

- The accuracy of the predictions at any time after loading was generally greater in the case of the concrete with the lower w/c ratio mix of each aggregate type, in comparison with the higher w/c ratio mix of that aggregate type;

Table 7.1 Summary of predicted specific strain magnitudes relative to the measured values

METHOD	Specific Creep Predictions							
	Specific Total Creep				Specific Basic Creep			
	Overpredicted	Underpredicted	Underpredicted then later Overpredicted	Overpredicted then later Underpredicted	Overpredicted	Underpredicted	Underpredicted then later Overpredicted	Overpredicted then later Underpredicted
BS 8110 (1985)		X						X
BS 8110 (1985) - modified		X						X
ACI 209 (1992)		X						X
AS 3600 (1988)		X				X		
CEB - FIP (1970)			X					X
CEB - FIP (1978)	X				X			
CEB - FIP (1990)		X				X		
RILEM Model B3 (1995)		X			X			

- The accuracy of the predictions does not increase with the complexity of the method applied or with increasing number of variables accounted for in the method.

The discrepancies between the results obtained by applying the different models appear to be due to the large uncertainty in the understanding of creep mechanisms which has led to the development of empirical models and not due to the failure to directly account for the aggregate stiffness in these methods.

According to Davis and Alexander (1992) the BS 8110 (1985) method, which has been incorporated into SABS 0100 (1992), does not make it possible to predict structural deformations with much precision. This was confirmed by the results of this investigation.

Furthermore, the magnitude of the aggregate stiffness is not reflected in any of the trends established and only in the case of the andesite concretes did one method, the CEB-FIP (1970) method, yield the most accurate predictions for both specific total and specific basic creep.

Comparison with results of other research projects

A comparison of the accuracy of the prediction methods included in this investigation with the accuracy determined for some of these methods by other researchers indicated the following:

- The results of this investigation showed the relative creep coefficients, which were established by Davis and Alexander (1992) with the intention of improving the accuracy of the BS 8110 (1985) method, to be inaccurate;

- The accuracy of the RILEM Model B3 (1995), ACI 209 (1992) and CEB-FIP (1990) established in this investigation is in fair agreement with the accuracy established for these models for total creep on the basis of data from the RILEM Data Bank (RILEM Model B3, 1995). However, significant disagreement exists when comparing the accuracy of the two sets of results for basic creep;

- McDonald et al., (1988) established the accuracy of the CEB-FIP (1978) and AS 3600 (1988) methods when predicting total creep only. The degree of accuracy of these predictions is in fair agreement with the results of this investigation;

- The accuracy determined by Alexander (1986) for the BS 8110 (1985) (previously C & CA, 1979 method), CEB-FIP (1970), CEB-FIP (1978) and ACI (1992/1982) methods is in general disagreement with the results of this investigation.

7.2 Recommendations for Further Research

7.2.1 Influence of aggregate type and stiffness on creep

In order to evaluate the true influence of aggregate type and stiffness on both basic and total creep strain, it is recommended that creep tests be conducted on sealed and unsealed specimens and that all the intrinsic and extrinsic factors except for the aggregate type be kept constant, as performed in this investigation. However, it is proposed that a constant quantity of condensed silica fume be included in all the samples to densify the aggregate/paste interfacial zone, thereby obtaining properties in this zone which are similar to those of the bulk hardened cement paste in the concrete. Mindess and Alexander, 1995 as well as Alexander and Milne (1995) found that the inclusion of silica fume in concrete densifies the interfacial zone between the paste and aggregate resulting in the interfacial zone being smaller and less compressible than it would have been if only OPC was used.

Furthermore, in the case of the proposed basic creep specimens, companion specimens should be cast (and sealed) for each mix and subjected to the same environment as the loaded sealed specimens. These companion specimens should be used for the purpose of recording the autogenous strains at the same times when total strains are measured on the loaded sealed specimens.

7.2.2 Further verification of the creep prediction models

The experimental creep results from the abovementioned proposed research should be compared with those predicted at the corresponding ages by the methods included in this investigation, in order to establish which method provides the most accurate estimates of creep strain of concrete made with either quartzite or granite or andesite aggregates.

Since the BS 8110 (1985) method is incorporated in the SABS 0100 (1992) code, the relative creep coefficients which were introduced by Davis and Alexander (1992) to improve the accuracy of the BS 8110 (1985) method, should be re-defined or confirmed. Should the accuracy of this method not be improved by further research, it is recommended that consideration be given to replacing the BS 8110 (1985) method in the SABS 0100 (1992) with a more accurate existing model, possibly the CEB-FIP (1970) model.

According to RILEM TC-107 (1995) basic creep and drying creep have different properties, originate from different mechanisms and depend on different variables. Hence basic creep and drying creep (in excess of the basic creep) should be represented by separate terms in a creep model, which may be added to obtain the total creep. The RILEM Model B3 (1995) is the only model, of those included in this investigation, that is structured in the manner described above. For this reason, particular attention should be paid to verifying the RILEM Model B3 (1995) and hence confirm this particular recommendation with regards to the structure of creep models.

APPENDIX A

DESIGN VALUES FOR ESTIMATING ELASTIC MODULUS OF CONCRETE

Table A.1 Design values for estimating elastic modulus of concrete for ages from 3 to 28 days (after Davis and Alexander, 1992)

AGGREGATE TYPE		DESIGN VALUES K_o	DESIGN VALUES α
CAPE			
Granite	(Rheebok)	21	0,25
Greywacke (Malmesbury Shale)	(Peninsula)	24	0,25
TM Quartzite	(Mossel Bay	23	0,25
NATAL			
Dolerite	(Leach and Brown)	20	0,40
Dolerite	(Natal Crushers)	18	0,30
Dolerite	(Ngagane)	22	0,40
TM Quartzite	(Coedmore)	19	0,30
TM Quartzite	(Verulam)	17	0,25
Tillite	(Umlaas Rd)	20	0,35
Siltstone	(Leach and Brown	21	0,15
TRANSVAAL			
Andesite	(Eikenhof)	29	0,20
Andesite	(Vaal)	24	0,35
Dolomite	(Sterkfontein)	25	0,45
Dolomite	(Olifantsfontein)	24	0,45
Felsite	(Middelburg)	18	0,35
Felsite	(Zeekoewater)	23	0,30
Granite	(Jukskei)	20	0,20
Granite	(Roodekrans)	15	0,30
Daspoort Quartzite	(Bundu)	14	0,30
Pretoria Quartzite	(Ferro)	17	0,40
Wits Quartzite	(DRD)	19	0,25
Wits Quartzite	(Scoops)	18	0,25
Wits Quartzite	(Vlakfontein	22	0,20

Note: Except for felsite, the range of cube strengths for which the above values are valid is generally between 20 and 70 MPa, at ages from 3 to 28 days. For strengths less than 20 MPa the expression will not be accurate. In the case of felsites the expression lacks accuracy below 25 MPa.

The number of tests conducted to determine the above values varied between 8 and 16. Each test result was the average of at least three individual specimens.

Table A.2 Design values for estimating elastic modulus of mature concretes of 6 months or older (after Davis and Alexander, 1992)

AGGREGATE TYPE		DESIGN VALUES	
		K_o	α
CAPE			
Granite	(Rheebok)	34	0,10
Greywacke (Malmesbury Shale)	(Peninsula)	31	0,20
TM Quartzite	(Mossel Bay	34	0,15
NATAL			
Dolerite	(Leach and Brown)	37	0,20
Dolerite	(Natal Crushers)	29	0,15
Dolerite	(Ngagane)	38	0,15
TM Quartzite	(Coedmore)	30	0,15
TM Quartzite	(Verulam)	32	0,10
Tillite	(Umlaas Rd)	29	0,20
Siltstone	(Leach and Brown	27	0,10
TRANSVAAL			
Andesite	(Eikenhof)	35	0,20
Andesite	(Vaal)	34	0,25
Dolomite	(Sterkfontein)	53	0,10
Dolomite	(Olifantsfontein)	41	0,25
Felsite	(Middelburg)	35	0,10
Felsite	(Zeekoewater)	27	0,25
Granite	(Jukskei)	31	0,10
Granite	(Roodekrans)	25	0,10
Daspoort Quartzite	(Bundu)	28	0,15
Pretoria Quartzite	(Ferro)	31	0,20
Wits Quartzite	(DRD)	34	0,10
Wits Quartzite	(Scoops)	33	0,10
Wits Quartzite	(Vlakfontein	35	0,10

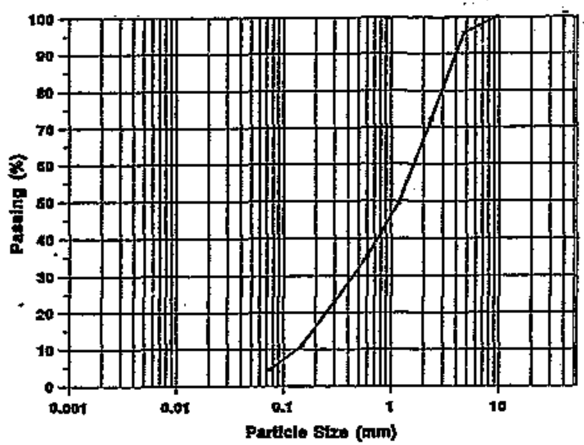
Note: The range of cube strengths for which the above values are valid is generally between 30 and 90 MPa, at the age of 6 months or later.

The number of tests conducted to determine the above values was 4 for each aggregate type. Each test result was the average of at least three individual specimens.

APPENDIX B

RESULTS OF GRADING ANALYSES ON CRUSHER SANDS

	Particle Size mm	% Passing
SIEVE	37,50	
	26,50	
	19,00	
	13,20	
	9,50	100
	4,75	95.8
	2,36	72.2
	1,18	49.5
	0,60	35.4
	0,425	
	0,300	23.0
	0,150	11.1
	0,075	4.7
HYDROMETER	0,075	
	0,050	
	0,040	
	0,026	
	0,015	
	0,010	
	0,0074	
	0,0036	
	0,0015	

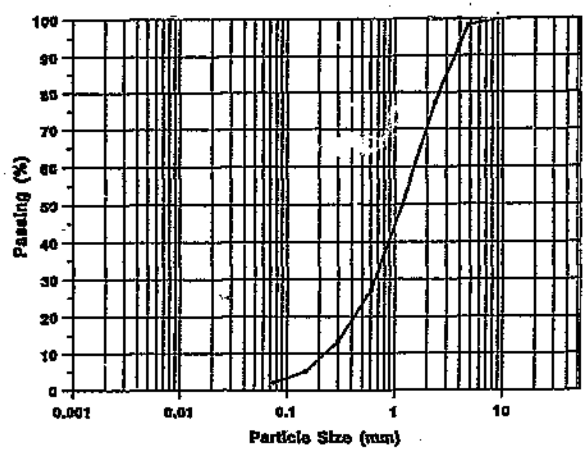


QUARTZITE STOCK PILE FERRO
JOB NUMBER : 187-847

FINENESS MODULUS = 3.09

Figure B.1 Results of grading analysis of quartzite crusher sand as received from the crusher

	Particle Size mm	% Passing
SIEVE	37,50	
	26,50	
	19,00	
	13,20	
	9,50	100
	4,75	98.2
	2,36	77.3
	1,18	50.0
	0,60	27.0
	0,425	
	0,300	13.4
	0,150	5.1
	0,075	2.0
HYDROMETER	0,075	
	0,050	
	0,040	
	0,026	
	0,015	
	0,010	
	0,0074	
	0,0036	
	0,0015	

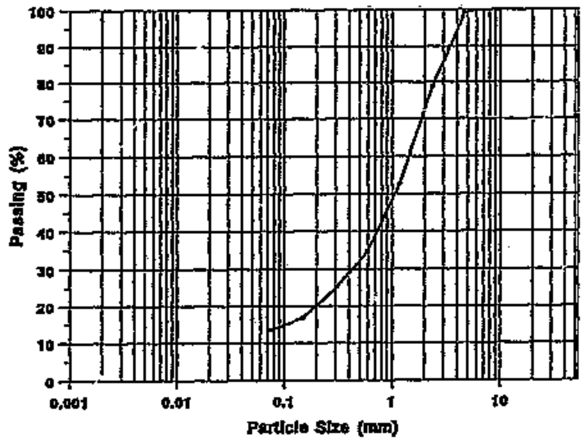


GRANITE STOCK PILE JUSKEI
JOB NUMBER : 182-789

FINENESS MODULUS = 2.92

Figure B.2 Results of grading analysis of granite crusher sand as received from the crusher

	Particle Size mm	% Passing
SIEVE	37,50	
	26,50	
	19,00	
	13,20	
	9,50	100
	4,75	99.8
	2,36	78.8
	1,18	52.8
	0,60	34.8
	0.425	
	0,300	24.8
	0,150	16.9
	0,076	13.4
HYDROMETER	0,075	
	0,050	
	0,040	
	0,026	
	0,015	
	0,010	
	0,0074	
	0,0036	
	0,0015	



ANDESITE STOCK PILE EIKENHOF
JOB NUMBER : 157-549

FINENESS MODULUS = 3.27

Figure B.3 Results of grading analysis of andesite crusher sand as received from the crusher

APPENDIX C

CUBE TEST RESULTS

Table C.1 Compressive strengths of the cubes of the different mixes

Mix No.	Compressive Strength Results			
	At 7 Days		At 28 Days	
	Individual (MPa)	Average (MPa)	Individual (MPa)	Average (MPa)
Q1	25.9* 26.5* 24.4*	25.6*	36.0 37.9 37.1	37.0
Q2	49.7* 51.2* 51.2*	50.7*	64.6 65.4 65.8	65.3
G1	24.9 24.0 24.3	24.4	37.2 37.8 38.0	37.7
G2	48.6 47.1 49.5	48.4	66.3 64.6 64.7	65.2
A1	34.3 33.4 35.4	34.4	48.1 45.6 50.5	48.1
A2	59.4 55.6 55.8	56.9	74.1 74.0 73.5	73.9

* Indicates test results determined at 8 days.

APPENDIX D

STRAIN MEASUREMENTS RECORDED ON TEST PRISMS

Table D.1 Strain measurements recorded on quartzite prisms (Mix Q1)

Age (Days)	Temp (Deg C)	RH (%)	LOADED SPECIMENS																UNLOADED SPECIMENS							
			Unsealed								Sealed															
			Specimen Number and Side						Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)	Specimen Number and Side						Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)	Specimen Number and Side						Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)
			1		2		3				1		2		3				1		2		3			
			A	B	A	B	A	B			A	B	A	B	A	B			A	B	A	B	A	B		
0	19	68	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		
0.007	19	68	267	434	267	518	234	451	362	362	217	431	200	434	251	384	323	323	0	0	0	0	0	0		0
1	15	70	134	117	84	50	100	117	100	462	67	117	84	134	117	67	97	420	50	50	50	17	17	50	39	39
2	19	60	33	17	17	17	50	17	25	487	17	-17	0	17	-17	0	0	420	-17	-17	-33	-33	-33	-17	-25	14
3	20	66	50	0	17	67	0	50	31	518	-67	-17	-17	0	-17	-33	-22	398	0	33	17	17	17	0	14	28
4	21	60	17	50	17	33	-17	33	22	540	-17	-33	0	-33	-17	-17	-19	379	-33	-17	-33	-67	-17	-33	-25	3
5	21	66	0	33	17	67	33	17	28	568	33	17	17	50	33	17	28	407	50	17	17	33	17	17	25	28
6	21	68	33	33	17	50	17	33	31	599	33	17	0	17	17	17	17	423	17	50	17	17	17	17	22	50
7	21	63	33	67	17	33	17	17	31	629	0	0	17	17	-17	17	6	429	0	-17	0	0	0	-17	-6	45
14	23	72	100	150	67	167	67	134	114	743	17	100	17	84	33	84	56	484	0	50	0	17	17	17	17	61
21	23	69	134	134	84	150	100	134	122	865	67	117	84	84	100	84	89	574	50	50	50	50	33	67	50	111
28	22	62	84	67	67	84	50	50	67	933	17	67	17	50	33	33	36	610	17	50	33	50	33	33	36	148
56	21	70	217	251	167	234	184	234	214	1147	100	217	117	134	167	150	148	757	67	67	67	33	67	67	61	209
112	22	63	134	117	100	117	84	134	114	1328	84	150	84	134	84	134	111	908	50	50	50	33	50	33	43	251
140	22	69	67	100	67	100	50	84	78	1406	50	100	50	100	100	84	81	988	17	50	33	50	17	50	36	287
168	25	72	0	0	0	17	0	17	6	1411	17	17	17	50	0	50	25	1013	-33	-33	-33	-50	-33	-33	-36	251

Table D.2 Strain measurements recorded on quartzite prisms (Mix Q2)

Age (Days)	Temp. (Deg. C)	RH (%)	LOADED SPECIMENS: Unsealed — Specimen Number and Side 1 A	1 B	2 A	2 B	3 A	3 B	Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)	LOADED SPECIMENS: Sealed — Specimen Number and Side 1 A	1 B	2 A	2 B	5 A	5 B	Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)	UNLOADED SPECIMENS — Specimen Number and Side 1 A	1 B	2 A	2 B	3 A	3 B	Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)
0	19	68	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		
0.003	19	68	468	501	451	506	451	518	482	482	452	401	500	434	518	418	454	454	0	0	0	0	0	0		0
1	15	70	134	117	134	117	100	150	125	607	134	67	150	84	100	84	103	557	33	33	33	33	33	33	33	33
2	19	68	0	50	17	17	0	50	22	630	-17	0	17	33	17	17	11	568	-33	-17	0	-17	17	-17	-11	22
3	20	66	33	50	17	67	33	33	39	668	17	33	17	17	67	17	28	596	33	0	17	0	0	0	8	30
4	23	60	33	0	0	17	-17	17	8	677	0	-33	-17	-17	17	-33	-14	582	-33	-17	-33	-17	-67	-67	-22	8
5	21	66	33	17	67	50	33	50	42	719	17	17	0	17	17	17	14	596	33	33	17	33	17	17	25	33
6	21	46	33	50	50	50	50	33	45	763	17	17	33	50	50	17	31	627	33	50	50	33	50	33	42	75
7	21	67	33	33	17	33	0	33	25	788	17	17	33	33	17	0	19	646	-17	-17	-17	0	-17	0	-11	64
14	23	72	150	134	134	184	134	134	145	933	33	50	50	117	67	67	64	710	33	50	33	17	33	33	33	97
21	22	69	134	100	117	150	150	134	131	1064	100	100	100	117	117	84	103	813	67	67	67	67	67	84	70	167
28	22	62	67	67	67	100	50	84	72	1136	50	84	67	67	50	50	61	874	50	33	50	50	67	50	50	217
56	21	70	184	184	200	200	184	200	192	1328	167	150	200	200	184	150	175	1050	67	67	84	67	50	67	67	284
112	23	61	84	100	100	117	84	117	100	1476	117	117	117	117	150	100	120	1224	67	50	67	50	67	50	58	331
140	22	69	100	67	100	84	84	84	86	1562	84	67	100	100	67	67	81	1295	33	50	33	33	33	50	39	370
168	25	72	-17	0	-17	0	-17	0	-8	1554	17	17	33	33	33	33	28	1323	-50	-67	-33	-33	-33	-50	-45	325

Table D.3 Strain measurements recorded on granite prisms (Mix G1)

Age (Days)	Temp (Deg. C)	RH (%)	LOADED SPECIMENS																UNLOADED SPECIMENS							
			Unsealed								Sealed															
			Specimen Number and Side						Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)	Specimen Number and Side						Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)	Specimen Number and Side						Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)
			1		2		3				1		2		3				1		2		3			
			A	B	A	B	A	B			A	B	A	B	A	B			A	B	A	B	A	B		
0	21	62	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		
0.007	21	62	200	351	267	334	317	334	284	284	301	363	317	267	367	267	315	315	0	0	0	0	0	0		0
1	20	66	84	184	100	150	134	167	136	420	50	67	84	67	67	50	64	379	0	-17	-17	-17	0	0	-8	-8
2	23	60	33	50	33	17	33	50	36	457	0	0	0	0	0	0	0	379	0	0	0	17	-17	17	3	-5
3	21	66	50	33	50	67	33	50	47	504	17	17	0	17	17	17	14	393	33	33	0	17	33	33	25	20
4	21	65	33	84	67	100	67	84	72	576	50	33	67	33	33	33	42	435	33	50	50	33	17	17	33	53
5	21	62	17	-17	-17	0	0	17	0	576	-33	0	-17	-33	0	0	-14	421	-17	0	-17	0	0	0	-6	48
6	21	66	17	50	33	33	33	33	33	610	0	17	17	17	0	0	8	429	17	0	0	0	0	0	3	50
7	22	63	17	17	17	17	17	33	19	629	0	0	17	0	33	0	8	437	0	17	0	17	-17	17	6	56
14	24	65	84	150	84	167	117	150	125	754	84	134	67	84	50	84	84	521	50	50	17	50	33	33	39	95
21	22	69	84	134	84	134	100	134	111	865	67	84	84	84	100	67	81	602	50	50	67	50	67	67	58	153
28	21	60	67	100	67	100	84	100	86	952	67	84	67	50	50	50	61	663	33	50	17	50	33	33	36	190
56	21	70	134	200	134	200	150	200	170	1122	100	150	134	117	134	117	125	788	67	84	67	84	50	84	72	262
112	21	74	117	134	100	134	134	134	125	1319	150	150	134	117	150	117	135	990	67	67	67	84	67	67	70	340
140	23	72	17	67	67	67	50	67	56	1375	33	67	67	67	67	67	63	1053	0	33	0	17	0	33	14	354
168	24	79	0	17	0	17	17	17	11	1386	17	50	17	17	50	33	33	1086	-17	-17	0	-17	0	-17	-11	343

Table D.4 Strain measurements recorded on granite prisms (Mix G2)

Age (Days)	Temp. (Deg. C)	RH (%)	LOADED SPECIMENS																UNLOADED SPECIMENS							
			Unsealed								Sealed															
			Specimen Number and Side						Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)	Specimen Number and Side						Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)	Specimen Number and Side						Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)
			1		2		3				1		2		3				1		2		3			
			A	B	A	B	A	B			A	B	A	B	A	B			A	B	A	B	A	B		
0	22	[illegible]	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		
0.007	22		501	468	468	501	518	484	490	490	551	501	518	518	518	501	518	518	0	0	0	0	0	0		0
1	20	66	[illegible]	150	167	184	150	184	167	657	117	84	84	100	117	100	100	618	17	17	33	0	0	0	11	11
2	23	[illegible]	[illegible]	50	50	84	67	50	61	718	17	17	33	0	17	0	14	633	0	0	0	17	0	0	3	14
3	21	66	[illegible]	67	67	67	67	67	70	788	33	33	33	33	33	33	33	666	33	33	33	17	17	33	28	42
4	21	66	[illegible]	67	67	67	50	67	64	852	50	33	33	50	50	33	42	707	50	33	50	50	50	50	47	89
5	21	62	17	17	17	0	17	17	14	866	-17	0	0	-17	0	0	-6	702	0	0	-17	-17	-17	-17	-11	78
6	21	66	17	33	33	50	17	30	33	[illegible]	17	33	0	17	17	17	17	718	17	17	0	17	0	17	11	89
7	23	63	33	17	17	33	33	17	25		0	17	0	0	17	17	8	727	0	0	0	0	0	0	0	86
14	24	65	167	117	150	134	117	134	136	[illegible]	[illegible]	134	84	117	117	100	111	838	67	67	33	33	33	33	45	133
21	22	69	134	117	134	134	134	134	131	1101	117	100	117	100	117	134	114	952	67	67	50	67	50	67	61	195
28	21	60	84	84	100	84	84	84	86	1278	67	67	84	50	67	50	64	1016	33	50	33	50	33	33	39	234
56	21	70	217	184	167	184	184	184	186	1464	184	200	167	184	184	200	186	1203	100	84	61	61	61	61	73	309
112	21	74	134	117	134	150	134	134	134	1659	150	150	150	134	150	150	148	1414	67	67	67	84	67	50	67	376
140	23	72	84	33	50	67	50	67	58	1717	50	50	50	33	50	50	47	1462	17	33	0	17	0	33	17	392
168	24	79	0	0	17	0	17	0	6	1723	67	33	50	33	33	33	41	1503	0	-17	0	0	0	-17	-6	387

Table D.5 Strain measurements recorded on andesite prisms (Mix A1)

Age (Days)	Temp. (Deg. C)	RH (%)	LOADED SPECIMENS																UNLOADED SPECIMENS							
			Unsealed								Sealed															
			Specimen Number and Side						Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)	Specimen Number and Side						Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)	Specimen Number and Side						Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)
			1		2		3				1		2		3				1		2		3			
			A	B	A	B	A	B			A	B	A	B	A	B			A	B	A	B	A	B		
0	23	60	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		
0.007	23	60	301	317	284	317	84	401	284	284	234	418	284	401	267	334	323	323	0	0	0	0	0	0		0
1	21	70	134	84	67	117	234	150	131	415	0	33	17	33	84	50	36	359	50	0	-17	17	17	0	11	11
2	21	66	84	67	84	67	50	67	70	484	33	33	17	67	17	33	33	393	17	17	17	50	33	17	25	36
3	21	62	17	0	0	0	0	0	3	487	0	0	0	0	17	0	3	395	0	-17	0	-17	0	-17	-8	28
4	21	66	50	33	33	50	17	50	39	526	0	17	0	17	0	33	11	407	0	0	0	17	17	0	6	33
5	22	63	33	17	17	17	17	17	19	546	0	17	17	17	0	33	14	420	0	17	0	0	0	0	3	36
6	22	63	17	17	17	17	17	17	17	562	0	17	17	17	17	0	11	432	33	0	17	0	17	0	11	47
7	23	62	33	17	33	17	17	33	25	587	17	17	17	33	17	50	25	457	0	0	-17	0	17	0	0	47
14	24	62	184	150	150	150	167	217	170	757	67	100	67	100	84	84	84	540	67	50	50	67	84	17	56	103
21	22	70	134	134	84	100	117	150	120	877	50	67	50	84	67	84	67	607	67	17	33	50	50	33	42	145
28	20	62	134	117	117	117	100	117	117	994	33	50	50	67	50	67	53	660	33	50	33	33	50	33	39	184
56	21	70	268	251	217	234	234	301	251	1244	117	167	117	184	167	200	159	818	100	67	84	84	100	67	84	267
112	22	70	117	134	100	117	100	134	117	1464	100	134	117	150	150	134	131	1033	50	50	33	67	50	50	50	334
140	21	78	67	84	50	67	50	100	70	1534	67	117	84	117	100	100	97	1130	33	50	33	33	33	17	33	367
168	23	76	33	17	17	17	17	17	19	1553	33	50	17	50	33	33	36	1166	-17	-33	-33	-33	-17	0	-22	345

Table D.6 Strain measurements recorded on andesite prisms (Mix A2)

Age (Days)	Temp. (Deg C)	RH (%)	LOADED SPECIMENS																UNLOADED SPECIMENS							
			Unsealed								Sealed															
			Specimen Number and Side						Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)	Specimen Number and Side						Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)	Specimen Number and Side						Average Incremental Strain (x10E-6)	Average Cumulative Strain (x10E-6)
			1		2		3				1		2		3				1		2		3			
			A	B	A	B	A	B			A	B	A	B	A	B			A	B	A	B	A	B		
0	23	60	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		
0.007	23	60	334	585	367	484	351	484	434	434	384	501	401	468	434	434	437	437	0	0	0	0	0	0		0
1	21	70	84	150	150	167	117	167	139	573	167	84	84	67	114	50	86	523	0	17	0	33	-17	17	8	8
2	21	66	84	84	100	100	100	100	95	668	50	33	67	33	50	33	45	568	17	33	150	50	17	50	53	61
3	21	62	-17	17	17	0	0	33	8	676	0	4	0	0	0	0	0	568	0	-17	33	0	17	0	6	66
4	21	60	67	33	50	50	33	67	50	726	33	50	17	17	17	17	25	593	0	17	17	33	17	0	14	80
5	22	63	17	33	33	17	0	17	19	746	17	33	0	17	17	17	17	610	0	0	0	-17	17	17	3	83
6	22	63	17	17	17	33	17	50	25	771	17	17	17	17	17	17	17	626	0	17	17	17	17	0	11	94
7	23	62	33	50	33	50	33	33	39	810	17	33	17	17	17	17	19	645	17	0	0	17	0	17	8	103
14	24	62	134	184	200	150	117	217	167	977	134	100	117	117	134	100	117	763	17	50	33	33	50	50	39	142
21	22	70	84	134	117	117	134	150	122	1099	84	84	84	84	84	84	84	846	33	50	50	17	50	50	42	183
28	20	62	84	117	117	100	134	117	111	1211	84	67	67	33	67	50	61	907	33	33	50	50	33	50	42	225
56	21	70	150	234	234	200	201	267	231	1442	200	200	184	184	217	167	192	1099	67	50	67	67	67	67	64	289
112	22	70	100	117	117	117	134	134	120	1645	134	150	150	100	134	117	131	1308	33	50	33	50	50	50	45	345
140	21	78	50	84	84	84	100	84	81	1725	84	84	67	84	100	67	81	1389	33	17	33	17	17	33	25	370
168	21	76	0	17	0	17	0	33	11	1737	33	33	33	17	50	33	33	1427	-33	-17	-33	-17	-17	-17	-22	348

APPENDIX E

AUTOGENOUS SHRINKAGE STRAINS

Table E.1 Inferred autogenous shrinkage strains

Mix	Alexander (1994b) Reference	Q1	Q2	G1	G2	A1	A2
a/c ratio	5.12	5.24	3.50	5.30	3.55	5.73	3.83
Factor (F)		0.98	1.46	0.97	1.44	0.89	1.34
Days (t)	Autogenous Shrinkage Strain						
	ε_{shR}	ε_{sho}					
0	0	0	0	0	0	0	0
1		12	18	12	18	11	17
2		24	36	24	36	22	33
3		37	54	36	54	33	50
4		49	72	48	72	44	66
5		61	91	60	90	55	83
6		73	109	72	107	66	100
7		85	127	84	125	77	116
10	124	122	181	120	179	110	166
14		138	205	136	202	124	188
21		165	246	163	243	150	226
28		193	287	191	283	175	264
30	205	201	299	199	295	182	275
56		249	371	247	366	226	341
100	337	330	492	327	485	300	452
112		339	506	336	499	308	465
140		361	538	358	531	328	494
168		383	570	379	563	348	523
200	416	408	607	404	599	370	557

APPENDIX F

RELATIVE CREEP COEFFICIENT DETAILS

Table F.1 Adjusted specific creep values, elastic moduli and relative creep coefficients

Measured							Davis and Alexander	
Mix No.	Adjustment Factor	Specific Total Creep at 168 Days			E (GPa)	Relative Creep	E (GPa)	Relative Creep
		Actual (x10E-6/ MPa)	Adjusted (x10E-6/ MPa)	Mean for Aggregate (x10E-6/ MPa)				
Q1	0.679	86.359	58.638					
Q2	1.193	45.733	54.559	56.599	73	0.92	70	0.96
G1	0.697	80.653	56.215					
G2	1.193	51.902	61.919	59.067	70	0.96	60	0.74
A1	0.880	76.997	67.757					
A2	1.358	51.699	70.207	68.982	89	1.12	95	1.19

APPENDIX G

SPECIFIC STRAINS OF TEST SPECIMENS

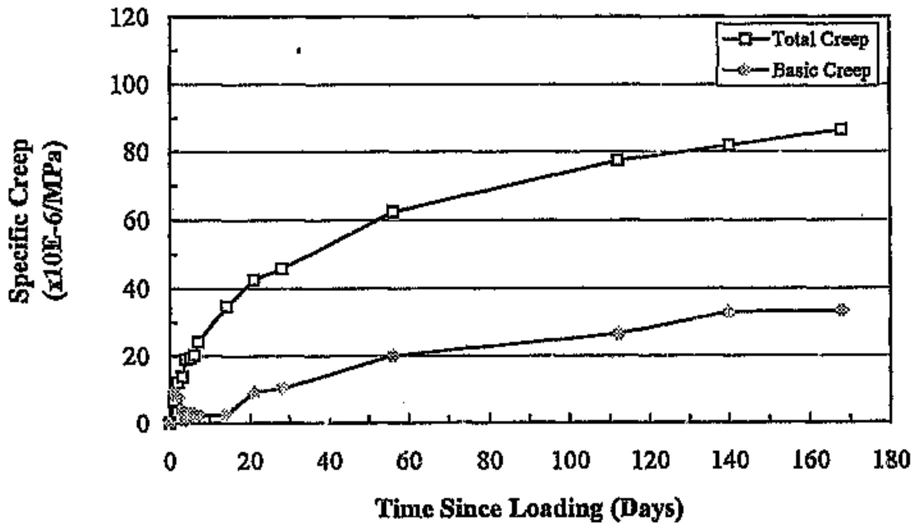


Figure G.1 Specific strains versus time since loading for mix Q1 specimens

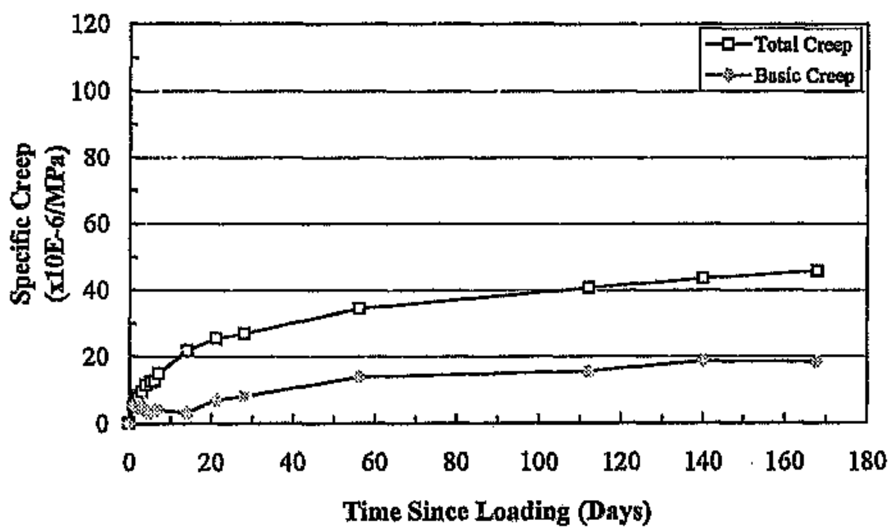


Figure G.2 Specific strains versus time since loading for mix Q2 specimens

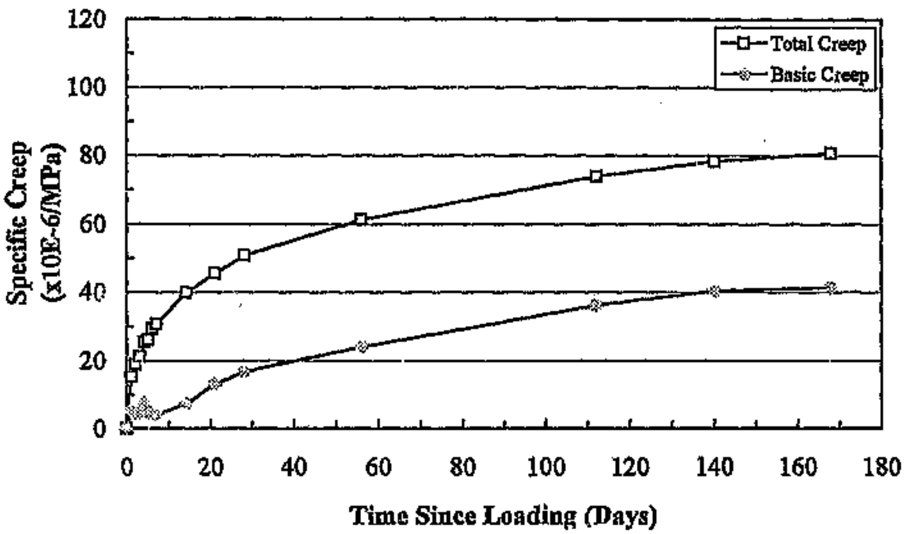


Figure G.3 Specific strains versus time since loading for mix G1 specimens

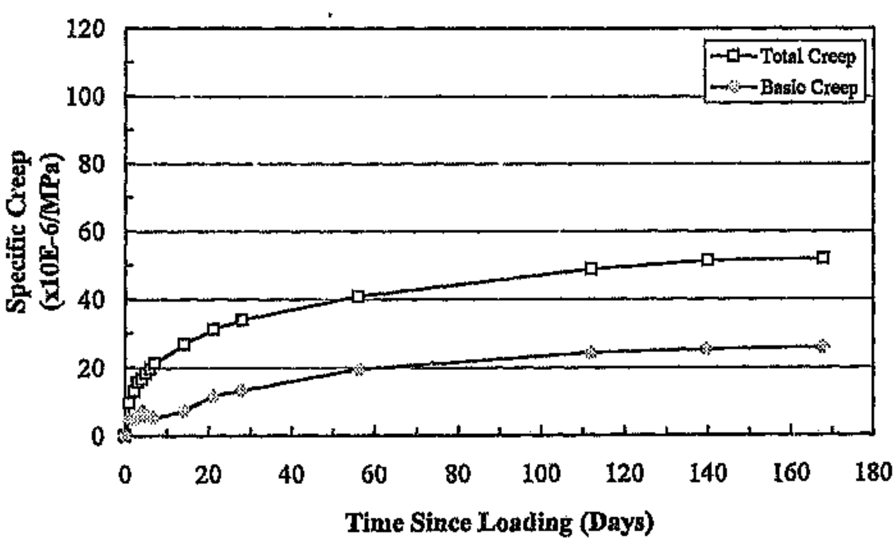


Figure G.4 Specific strains versus time since loading for mix G2 specimens

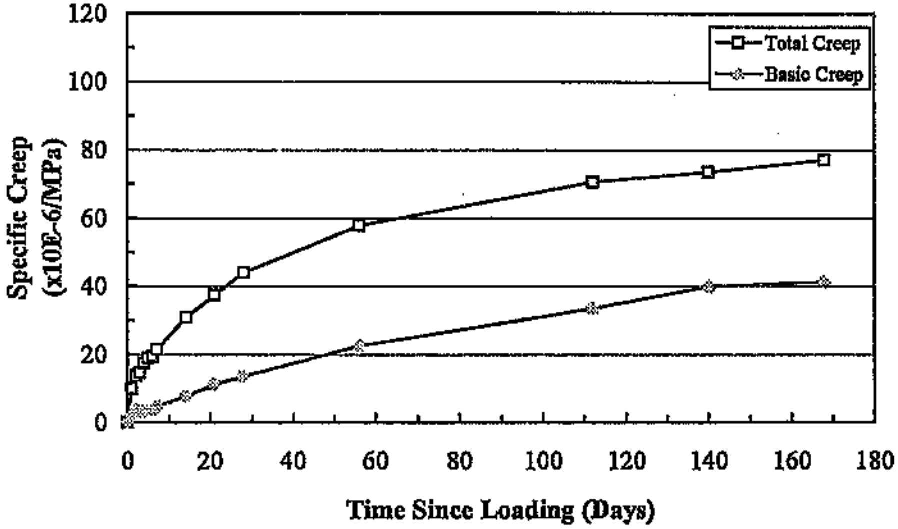


Figure G.5 Specific strains versus time since loading for mix A1 specimens

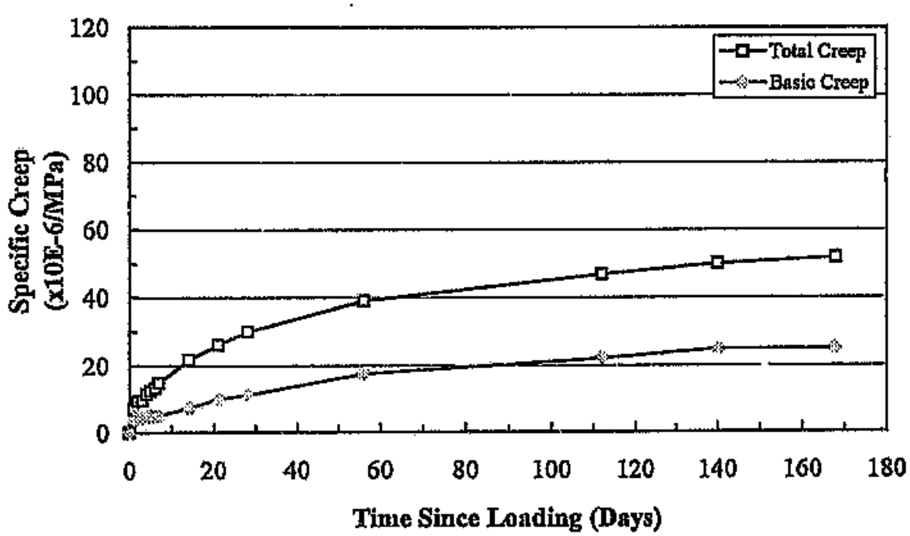


Figure G.6 Specific strains versus time since loading for mix A2 specimens

APPENDIX H

VARIABLES USED FOR CREEP PREDICTIONS

Table H.1 Variables used for creep predictions according to the BS 8110 (1985) and BS 8110 (1985) - Modified methods

Variables	Mix No.					
	Q1	Q2	G1	G2	A1	A2
σ (MPa)	9.25	16.30	9.42	16.30	12.00	18.47
f_{cu} cube (MPa)	37	65	38	65	48	74
K_0 (GPa)	17	17	20	20	29	29
α (GPa/MPa)	0.4	0.4	0.2	0.2	0.2	0.2
E (GPa)	31.8	43.0	27.6	33.0	38.6	43.8
ϕ^* Total	2.4	2.4	2.4	2.4	2.4	2.4
ϕ^* Basic	1.7	1.7	1.7	1.7	1.7	1.7
ϕ^* Total (Modified)	2.28	2.28	1.78	1.78	2.86	2.86
ϕ^* Basic (Modified)	1.62	1.62	1.26	1.26	2.02	2.02
Relative Creep Coefficient	0.95	0.95	0.74	0.74	1.19	1.19

Table H.2 Calculated ratio of final creep assumed to develop at different loading ages (BS 8110, 1985)

Age (Days)	Ratio of Final Creep
0	0
1	0.029
2	0.106
3	0.152
4	0.184
5	0.209
6	0.229
7	0.247
14	0.324
21	0.370
28	0.402
56	0.479
112	0.557
140	0.582
168	0.603

Table H.3 Variables used for creep predictions according to the ACI 209 (1992) method

Variables	Mix No.					
	Q1	Q2	G1	G2	A1	A2
σ (MPa)	9.25	16.30	9.42	16.30	12.00	18.47
f_{cu} cube (MPa)	37	65	38	65	48	74
f_{cu} cyl. (MPa)	30	55	31	55	38.5	62.7
E (GPa)	27.691	37.494	28.149	37.494	35.372	45.140
$\rho_{concrete}$ (kg/m^3)	2400	2400	2400	2400	2600	2600
Slump (mm)	90	50	115	70	95	55
ψ	43.7	39.8	47.7	44.2	43.1	39.2
γ_1 (for $\tau = 28$ days)	0.844	0.844	0.844	0.844	0.844	0.844
γ_2 Total creep	0.835	0.835	0.835	0.835	0.835	0.835
γ_2 Basic creep	0.6	0.6	0.6	0.6	0.6	0.6
γ_3 (for $h_o = 100$mm)	1.11	1.11	1.11	1.11	1.11	1.11
γ_4	1.058	0.952	1.124	1.005	1.071	0.965
γ_5	0.985	0.976	0.994	0.986	0.983	0.974
γ_6	1	1	1	1	1	1
ϕ^* Total	1.915	1.708	2.053	1.821	1.935	1.728
ϕ^* Basic	1.376	1.227	1.475	1.309	1.390	1.242

Table H.4 Variables used for creep predictions according to the AS 3600 (1988) method

Variables	Mix No.		
	Q1	G1	A1
σ (MPa)	9.25	9.42	12.00
f_{cu} cube (MPa)	37	38	48
f_{cu} cyl. (MPa)	30	31	38.5
E (GPa)	27.691	28.149	35.372
$\rho_{concrete}$ (kg/m^3)	2400	2400	2600
$\phi_{cc.b}$	2.8	2.7	2.1
k_3	1.1	1.1	1.1
t_h (mm) Total creep	50	50	50
t_h (mm)Basic creep	400	400	400
Environment	Tropical and Near Coastal		

Table H.5 Creep factor coefficients (k_2) for different ages after loading (after AS 3600, 1988)

Age (Days)	Creep Factor Coefficient (k_2)	
	Total Creep	Basic Creep
1	0.09	0.01
2	0.12	0.01
3	0.16	0.02
4	0.18	0.02
5	0.20	0.02
6	0.22	0.02
7	0.24	0.02
14	0.34	0.04
21	0.39	0.05
28	0.47	0.06
56	0.54	0.08
112	0.66	0.12
140	0.67	0.12
168	0.68	0.14

Table H.6 Variables used for creep predictions according to the CEB-FIP (1970) method

Variables	Mix No.					
	Q1	Q2	G1	G2	A1	A2
σ (MPa)	9.25	16.30	9.42	16.30	12.00	18.47
f_{cu} cube (MPa)	37	65	38	65	48	74
f_{cu} cyl. (MPa)	30	55	31	55	38.5	62.7
E (GPa)	32.534	44.052	33.073	44.052	36.857	47.035
k_1 Total creep	2.38	2.38	2.38	2.38	2.38	2.38
k_1 Basic creep	1	1	1	1	1	1
k_2	1.004	1.004	1.004	1.004	1.004	1.004
k_3 (for w/c = 0.56)	1.242	1.242	1.242	1.242	1.242	1.242
k_3 (for w/c = 0.4)	1.035	1.035	1.035	1.035	1.035	1.035
k_4	1.191	1.191	1.191	1.191	1.191	1.191
b (mm)	50	50	50	50	50	50

Table H.7 Calculated k_5 coefficients for different loading ages (after CEB-FIP, 1970)

Age (Days)	k_5
0	0
1	0.074
2	0.122
3	0.162
4	0.195
5	0.225
6	0.251
7	0.275
14	0.398
21	0.477
28	0.535
56	0.667
112	0.777
140	0.807
168	0.828

Table H.8 Variables used for creep predictions according to the CEB-FIP (1978) method

Variables	Mix No.					
	Q1	Q2	G1	G2	A1	A2
σ (MPa)	9.25	16.30	9.42	16.30	12.00	18.47
f_{cu} cube (MPa)	37	65	38	65	48	74
f_{cu} cyl. (MPa)	30	55	31	55	38.5	62.7
E (GPa)	29.519	36.128	29.843	36.128	32.078	37.741
ϕ_d	0.4	0.4	0.4	0.4	0.4	0.4
$\beta_a(\tau)$	0.253	0.253	0.253	0.253	0.253	0.253

Table H.9 Flow coefficient variables used for creep predictions according to the CEB-FIP (1978) method

Creep Type	Variable						
	ϕ_{f1}	ϕ_{f2}	ϕ_f	λ	A_c (mm^2)	u	h_o
Total Creep	2.278	1.8	4.078	1.33	1.00E+04	400	66.5
Basic Creep	0.8	1.173	1.973	30	1.00E+04	400	1500

Table H.10 Calculated values of the $\beta_d(t-\tau)$ and $\beta_f(t) - \beta_f(\tau)$ functions for different ages of loading (CEB-FIP, 1978)

Duration of Load (Days)	$\beta_d(t-\tau)$	$\beta_f(t) - \beta_f(\tau)$ Total Creep	Basic Creep
1	0.277	0.102	0.109
2	0.284	0.137	0.131
3	0.291	0.162	0.146
4	0.299	0.182	0.158
5	0.306	0.200	0.168
6	0.313	0.216	0.176
7	0.319	0.230	0.184
14	0.365	0.305	0.221
21	0.408	0.359	0.246
28	0.448	0.402	0.265
56	0.583	0.522	0.318
112	0.762	0.656	0.380
140	0.820	0.699	0.402
168	0.864	0.734	0.421

Table H.11 Variables used for creep predictions according to the CEB-FIP (1990) method

Variables	Mix No.					
	Q1	Q2	G1	G2	A1	A2
σ (MPa)	9.25	16.30	9.42	16.30	12.00	18.47
f_{cu} cube (MPa)	37	65	38	65	48	74
f_{cu} cyl. (MPa)	30	55	31	55	38.5	62.7
E (GPa)	31.072	38.030	31.414	38.030	33.767	39.727
ϕ_{RH} Total creep	1.95	1.95	1.95	1.95	1.95	1.95
ϕ_{RH} Basic creep	1	1	1	1	1	1
A_c (mm^2)	10000	10000	10000	10000	10000	10000
u (mm)	400	400	400	400	400	400
h_o (mm)	50	50	50	50	50	50
$\beta(f_{cm})$	2.762	2.084	2.725	2.084	2.425	1.953
t_o (days)	28	28	28	28	28	28
β (t_o)	0.4885	0.4885	0.4885	0.4885	0.4885	0.4885
ϕ_o Total creep	2.630	1.985	2.595	1.985	2.310	1.860
ϕ_o Basic creep	1.349	1.018	1.331	1.018	1.185	0.954
β_H Total creep	326	326	326	326	326	326
β_H Basic creep	1500	1500	1500	1500	1500	1500

Table H.12 Calculated values of the $\beta_c(t - t_o)$ function for different ages of loading (after CEB-FIP, 1990)

Duration of Load (Days)	$\beta_c(t - t_o)$	
	Total Creep	Basic Creep
1	0.176	0.111
2	0.217	0.137
3	0.244	0.155
4	0.266	0.169
5	0.284	0.180
6	0.300	0.191
7	0.314	0.200
14	0.384	0.245
21	0.431	0.277
28	0.467	0.301
56	0.562	0.369
112	0.664	0.449
140	0.697	0.478
168	0.724	0.502

Table H.13 Variables used for creep predictions according to the RILEM Model B3 (1995) method

Creep Type	Variables	Mix No.					
		Q1	Q2	G1	G2	A1	A2
General	σ (MPa)	9.25	16.30	9.42	16.30	12.00	18.47
	f_{cu} cube (MPa)	37	65	38	65	48	74
	f_{cu} cyl. (MPa)	30	55	31	55	38.5	62.7
	E (τ) (GPa)	25.924	35.101	26.352	35.101	29.367	37.477
Basic creep	m	0.5	0.5	0.5	0.5	0.5	0.5
	n	0.1	0.1	0.1	0.1	0.1	0.1
	c (kg/m^3)	348.2	487.5	348.2	487.5	348.2	487.5
	w/c	0.56	0.40	0.56	0.40	0.56	0.40
	a/c	5.24	3.51	5.30	3.54	5.73	3.83
	q_2	1.1170	0.7659	1.0845	0.7659	0.8923	0.6807
	q_3	0.03186	0.00569	0.03093	0.00569	0.02545	0.00505
	q_4	0.04391	0.05813	0.04356	0.05779	0.04125	0.05469
	r (τ)	10.536	10.536	10.536	10.536	10.536	10.536
	Q_f (τ)	0.1818	0.1818	0.1818	0.1818	0.1818	0.1818
Drying creep	t_o (days)	28	28	28	28	28	28
	k_t	17.995	15.465	17.848	15.465	16.907	14.966
	k_s	1.25	1.25	1.25	1.25	1.25	1.25
	D (mm)	50	50	50	50	50	50
	τ_{sh}	108.950	93.640	108.060	93.640	102.370	90.610
	w (kg/m^3)	195	195	195	195	195	195
	$\varepsilon_{s\infty}$	743.061	669.218	738.738	669.218	711.146	654.837

Table H.14 Calculated values of the Z (t, τ) and Q (t, τ) functions for different ages of loading required for the prediction of basic creep (RILEM Model B3, 1995)

Days (t - τ)	Function Z (t, τ)	Function Q (t, τ)
1	0.131	0.131
2	0.138	0.137
3	0.142	0.141
4	0.145	0.143
5	0.147	0.145
6	0.149	0.147
7	0.150	0.148
14	0.158	0.155
21	0.162	0.158
28	0.165	0.160
56	0.173	0.165
112	0.181	0.170
140	0.183	0.171
168	0.186	0.172

Table H.15 Calculated values of the E (t) for different ages of loading required for the prediction of drying creep (RILEM Model B3, 1995)

Days (t)	E (t) (MPa)					
	Mix QI	Mix Q2	Mix G1	Mix G2	Mix A1	Mix A2
29	26082	35315	26512	35315	29546	37705
30	26143	35397	26574	35397	29615	37793
31	26200	35475	26633	35475	29680	37876
32	26254	35548	26688	35548	29741	37954
33	26305	35617	26740	35617	29799	38028
34	26354	35683	26789	35683	29854	38098
35	26400	35745	26836	35745	29906	38165
42	26664	36103	27105	36103	30206	38547
49	26858	36366	27302	36366	30425	38828
56	27007	36567	27453	36567	30593	39042
112	27546	37297	28001	37297	31204	39822
140	27658	37448	28114	37448	31331	39983
168	27733	37550	28191	37550	31416	40092
196	27787	37623	28246	37623	31477	40170

Table H.16 Calculated values of $\varepsilon_{sh\infty}$, S(t), H(t) and q_5 at different ages of loading for quartzite-aggregate concretes (RILEM Model B3, 1995)

Duration of Load $(t - \tau)$ (Days)	Mix No.							
	Q1				Q2			
	$\varepsilon_{sh\infty}$	S(t)	H(t)	q_5	$\varepsilon_{sh\infty}$	S(t)	H(t)	q_5
1	3273.509	0.096	0.967	1.354	3319.265	0.103	0.964	0.732
2	3265.883	0.135	0.953	1.356	3311.532	0.145	0.949	0.733
3	3258.732	0.164	0.942	1.357	3304.282	0.177	0.938	0.734
4	3252.015	0.189	0.934	1.359	3297.471	0.204	0.929	0.735
5	3245.692	0.211	0.926	1.361	3291.059	0.227	0.921	0.736
6	3239.729	0.230	0.919	1.362	3285.013	0.248	0.913	0.737
7	3234.097	0.248	0.913	1.364	3279.303	0.267	0.907	0.738
14	3201.996	0.344	0.880	1.372	3246.753	0.368	0.871	0.742
21	3178.868	0.413	0.856	1.378	3223.302	0.441	0.846	0.745
28	3161.412	0.468	0.836	1.382	3205.601	0.498	0.826	0.748
56	3120.299	0.615	0.785	1.393	3163.914	0.649	0.773	0.754
112	3087.015	0.767	0.731	1.402	3130.165	0.798	0.721	0.758
140	3078.638	0.812	0.716	1.404	3121.670	0.840	0.706	0.760
168	3072.640	0.846	0.704	1.406	3115.589	0.872	0.695	0.761

Table H.17 Calculated values of $\varepsilon_{sh\infty}$, S(t), H(t) and q_5 at different ages of loading for granite-aggregate concretes (RILEM Model B3, 1995)

Duration of Load $(t - \tau)$ (Days)	Mix No.							
	G1				G2			
	$\varepsilon_{sh\infty}$	S(t)	H(t)	q_5	$\varepsilon_{sh\infty}$	S(t)	H(t)	q_5
1	3275.751	0.096	0.966	1.309	3319.265	0.103	0.964	0.732
2	3268.119	0.135	0.953	1.311	3311.532	0.145	0.949	0.733
3	3260.964	0.165	0.942	1.313	3304.282	0.177	0.938	0.734
4	3254.242	0.190	0.933	1.315	3297.471	0.204	0.929	0.735
5	3247.914	0.212	0.926	1.316	3291.059	0.227	0.921	0.736
6	3241.948	0.231	0.919	1.318	3285.013	0.248	0.913	0.737
7	3236.312	0.249	0.913	1.319	3279.303	0.267	0.907	0.738
14	3204.189	0.345	0.879	1.327	3246.753	0.368	0.871	0.742
21	3181.046	0.414	0.855	1.333	3223.302	0.441	0.846	0.745
28	3163.577	0.469	0.836	1.337	3205.601	0.498	0.826	0.748
56	3122.436	0.617	0.784	1.348	3163.914	0.649	0.773	0.754
112	3089.129	0.769	0.731	1.356	3130.165	0.798	0.721	0.758
140	3080.746	0.814	0.715	1.359	3121.670	0.840	0.706	0.760
168	3074.744	0.847	0.703	1.360	3115.589	0.872	0.695	0.76[illegible]

Table H.18 Calculated values of $\varepsilon_{sh\infty}$, S(t), H(t) and q_5 at different ages of loading for andesite-aggregate concretes (RILEM Model B3, 1995)

Duration of Load (t - τ) (Days)	Mix No.							
	A1				A2			
	$\varepsilon_{sh\infty}$	S(t)	H(t)	q_5	$\varepsilon_{sh\infty}$	S(t)	H(t)	q_5
1	3291.031	0.099	0.966	1.051	3330.908	0.105	0.963	0.641
2	3283.365	0.139	0.951	1.053	3323.148	0.147	0.948	0.642
3	3276.176	0.170	0.941	1.054	3315.872	0.180	0.937	0.643
4	3269.422	0.195	0.932	1.056	3309.037	0.207	0.928	0.643
5	3263.065	0.217	0.924	1.057	3302.603	0.231	0.919	0.644
6	3257.071	0.237	0.917	1.058	3296.536	0.252	0.912	0.645
7	3251.409	0.256	0.911	1.059	3290.805	0.271	0.905	0.646
14	3219.136	0.354	0.876	1.065	3258.141	0.374	0.869	0.650
21	3195.884	0.424	0.851	1.070	3234.608	0.447	0.843	0.652
28	3178.334	0.480	0.832	1.074	3216.845	0.505	0.823	0.654
56	3137.002	0.629	0.780	1.082	3175.012	0.656	0.770	0.660
112	3103.539	0.780	0.727	1.089	3141.144	0.805	0.718	0.664
140	3095.117	0.824	0.712	1.091	3132.620	0.846	0.704	0.665
168	3089.087	0.857	0.700	1.092	3126.517	0.877	0.693	0.666

APPENDIX I

MEASURED AND PREDICTED CREEP COEFFICIENTS

Table I.1 Measured and predicted total creep coefficients and basic creep coefficients for different ages of loading for mix Q1 concrete specimens

Duration of Load (Days)	Creep Coefficient ϕ (Creep Factor)																	
	Total Creep									Basic Creep								
	Measured	Predicted								Measured	Predicted							
		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	AS 3600 (1988)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	AS 3600 (1988)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)
1	0.161	0.069	0.065	0.174	0.277	0.262	0.782	0.463	0.689	0.252	0.049	0.046	0.125	0.031	0.110	0.579	0.150	0.637
2	0.293	0.255	0.242	0.252	0.370	0.431	0.925	0.570	0.737	0.217	0.181	0.172	0.181	0.031	0.181	0.626	0.185	0.674
3	0.337	0.364	0.346	0.310	0.493	0.573	1.030	0.643	0.769	0.113	0.258	0.246	0.223	0.062	0.241	0.639	0.209	0.698
4	0.461	0.441	0.419	0.358	0.554	0.689	1.116	0.700	0.794	0.020	0.313	0.298	0.237	0.062	0.290	0.684	0.228	0.716
5	0.469	0.501	0.476	0.398	0.616	0.795	1.191	0.748	0.815	0.066	0.355	0.338	0.286	0.062	0.334	0.706	0.244	0.731
6	0.491	0.550	0.523	0.434	0.678	0.887	1.257	0.789	0.833	0.080	0.390	0.371	0.312	0.062	0.373	0.726	0.257	0.744
7	0.586	0.592	0.562	0.466	0.739	0.972	1.318	0.826	0.850	0.061	0.419	0.399	0.335	0.062	0.408	0.743	0.269	0.756
14	0.842	0.778	0.739	0.627	1.047	1.407	1.643	1.010	0.936	0.069	0.551	0.525	0.451	0.123	0.591	0.835	0.331	0.816
21	1.033	0.887	0.843	0.734	1.201	1.686	1.880	1.134	0.999	0.252	0.628	0.599	0.527	0.154	0.708	0.902	0.373	0.859
28	1.113	0.964	0.916	0.813	1.448	1.891	2.072	1.229	1.049	0.276	0.683	0.651	0.585	0.185	0.795	0.956	0.406	0.892
56	1.516	1.151	1.093	1.011	1.663	2.358	2.614	1.479	1.191	0.546	0.815	0.777	0.727	0.246	0.991	1.114	0.498	0.986
112	1.882	1.337	1.270	1.205	2.033	2.747	3.233	1.748	1.359	0.724	0.947	0.903	0.866	0.370	1.154	1.308	0.606	1.093
140	1.992	1.397	1.327	1.264	2.064	2.853	3.433	1.834	1.416	0.897	0.990	0.943	0.908	0.370	1.199	1.375	0.645	1.130
168	2.102	1.446	1.374	1.310	2.094	2.927	3.591	1.904	1.462	0.906	1.024	0.976	0.941	0.431	1.230	1.430	0.678	1.161

Table L2 Measured and predicted total creep coefficients and basic creep coefficients for different ages of loading for mix Q2 concrete specimens

Duration of Load (Days)	Creep Coefficient ϕ (Creep Factor)															
	Total Creep								Basic Creep							
	Measured	Predicted							Measured	Predicted						
		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)
1	0.185	0.069	0.065	0.155	0.218	0.782	0.349	0.579	0.182	0.049	0.046	0.112	0.092	0.579	0.113	0.540
2	0.252	0.255	0.242	0.225	0.359	0.925	0.430	0.624	0.167	0.181	0.172	0.162	0.151	0.626	0.140	0.576
3	0.314	0.364	0.346	0.277	0.477	1.030	0.485	0.655	0.188	0.258	0.246	0.199	0.201	0.659	0.158	0.601
4	0.376	0.441	0.419	0.319	0.574	1.116	0.528	0.681	0.120	0.313	0.298	0.229	0.241	0.684	0.172	0.621
5	0.409	0.501	0.476	0.355	0.663	1.191	0.564	0.703	0.109	0.355	0.338	0.255	0.279	0.706	0.184	0.638
6	0.415	0.550	0.523	0.387	0.739	1.257	0.596	0.723	0.136	0.390	0.371	0.278	0.311	0.726	0.194	0.654
7	0.488	0.592	0.562	0.415	0.810	1.318	0.623	0.740	0.139	0.419	0.399	0.299	0.340	0.743	0.203	0.668
14	0.713	0.778	0.739	0.560	1.173	1.643	0.762	0.840	0.109	0.551	0.525	0.402	0.493	0.835	0.250	0.747
21	0.837	0.887	0.843	0.655	1.405	1.880	0.856	0.915	0.242	0.628	0.599	0.470	0.590	0.902	0.282	0.807
28	0.882	0.964	0.916	0.726	1.576	2.072	0.927	0.977	0.285	0.683	0.651	0.521	0.662	0.956	0.307	0.856
56	1.135	1.151	1.093	0.902	1.965	2.614	1.116	1.156	0.481	0.815	0.777	0.648	0.826	1.114	0.376	0.997
112	1.337	1.337	1.270	1.075	2.289	3.233	1.319	1.370	0.544	0.947	0.903	0.772	0.962	1.308	0.457	1.166
140	1.433	1.397	1.327	1.127	2.377	3.433	1.384	1.443	0.649	0.990	0.943	0.810	0.999	1.375	0.487	1.225
168	1.506	1.446	1.374	1.168	2.439	3.591	1.436	1.504	0.640	1.024	0.976	0.839	1.025	1.430	0.511	1.274

Table I.3 Measured and predicted total creep coefficients and basic creep coefficients for different ages of loading for mix G1 concrete specimens

Duration of Load (Days)	Creep Coefficient φ (Creep Factor)																	
	Total Creep									Basic Creep								
	Measured	Predicted								Measured	Predicted							
		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	AS 3600 (1988)	CEB-FIP (1970)	CEB-FIP (1978)	CEB FIP (1990)	RILEM Model B3 (1995)		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	AS 3600 (1988)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)
1	0.448	0.069	0.051	0.187	0.267	0.262	0.782	0.457	0.689	0.144	0.049	0.036	0.134	0.030	0.110	0.579	0.148	0.629
2	0.552	0.255	0.189	0.270	0.356	0.431	0.925	0.562	0.728	0.111	0.181	0.134	0.194	0.030	0.181	0.626	0.183	0.665
3	0.621	0.364	0.270	0.333	0.475	0.573	1.030	0.634	0.759	0.116	0.258	0.191	0.239	0.059	0.241	0.659	0.206	0.689
4	0.742	0.441	0.327	0.384	0.535	0.689	1.116	0.691	0.784	0.199	0.313	0.232	0.276	0.059	0.290	0.684	0.225	0.707
5	0.759	0.501	0.372	0.427	0.594	0.795	1.191	0.738	0.805	0.127	0.355	0.263	0.307	0.059	0.334	0.706	0.240	0.722
6	0.854	0.550	0.408	0.465	0.653	0.887	1.257	0.779	0.823	0.117	0.390	0.289	0.334	0.059	0.373	0.726	0.254	0.735
7	0.898	0.592	0.439	0.499	0.713	0.972	1.318	0.815	0.839	0.106	0.419	0.311	0.359	0.059	0.408	0.743	0.266	0.747
14	1.166	0.778	0.577	0.673	1.010	1.407	1.643	0.997	0.925	0.195	0.551	0.409	0.483	0.119	0.591	0.835	0.327	0.807
21	1.330	0.887	0.658	0.787	1.158	1.686	1.880	1.119	0.987	0.345	0.628	0.466	0.565	0.149	0.708	0.902	0.368	0.849
28	1.485	0.964	0.715	0.872	1.396	1.891	2.072	1.213	1.038	0.438	0.683	0.506	0.627	0.178	0.793	0.956	0.401	0.883
56	1.788	1.151	0.853	1.084	1.604	2.358	2.614	1.459	1.180	0.632	0.815	0.604	0.779	0.238	0.991	1.114	0.491	0.977
112	2.160	1.337	0.992	1.292	1.960	2.747	3.233	1.724	1.347	0.952	0.947	0.702	0.928	0.356	1.154	1.308	0.598	1.085
140	2.289	1.397	1.036	1.335	1.990	2.853	3.433	1.810	1.403	1.062	0.990	0.733	0.973	0.356	1.199	1.375	0.636	1.122
168	2.358	1.446	1.073	1.404	2.020	2.927	3.591	1.878	1.449	1.089	1.024	0.759	1.009	0.416	1.230	1.430	0.669	1.152

Table I.4 Measured and predicted total creep coefficients and basic creep coefficients for different ages of loading for mix G2 concrete specimens

Duration of Load (Days)	Creep Coefficient φ (Creep Factor)															
	Total Creep								Basic Creep							
	Measured	Predicted							Measured	Predicted						
		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)
[illegible]	0.284	0.069	0.051	0.166	0.218	0.782	0.349	0.579	0.141	0.049	0.036	0.119	0.092	0.579	0.113	0.540
2	0.391	0.255	0.189	0.240	0.359	0.925	0.430	0.624	0.134	0.181	0.134	0.172	0.151	0.626	0.140	0.575
3	0.467	0.364	0.270	0.295	0.477	1.030	0.485	0.655	0.160	0.258	0.191	0.212	0.201	0.659	0.158	0.600
4	0.498	0.441	0.327	0.340	0.574	1.116	0.528	0.681	0.202	0.313	0.232	0.244	0.241	0.684	0.172	0.620
5	0.543	0.501	0.372	0.379	0.663	1.191	0.564	0.703	0.160	0.355	0.263	0.272	0.279	0.706	0.184	0.638
6	0.584	0.550	0.408	0.413	0.739	1.257	0.596	0.722	0.160	0.390	0.289	0.297	0.311	0.726	0.194	0.653
7	0.630	0.592	0.439	0.443	0.810	1.318	0.623	0.740	0.143	0.419	0.311	0.318	0.340	0.743	0.203	0.668
14	0.797	0.778	0.577	0.597	1.173	1.643	0.762	0.839	0.203	0.551	0.409	0.429	0.493	0.835	0.250	0.746
21	0.924	0.887	0.658	0.698	1.405	1.880	0.856	0.914	0.329	0.628	0.466	0.502	0.590	0.902	0.282	0.806
28	1.010	0.964	0.715	0.774	1.576	2.072	0.927	0.976	0.371	0.683	0.506	0.556	0.662	0.956	0.307	0.854
56	1.214	1.151	0.853	0.962	1.965	2.614	1.114	1.154	0.548	0.815	0.604	0.691	0.826	1.114	0.376	0.995
112	1.447	1.337	0.992	1.146	2.289	3.233	1.319	1.367	0.684	0.947	0.702	0.823	0.962	1.308	0.457	1.163
140	1.523	1.397	1.036	1.202	2.377	3.433	1.384	1.440	0.710	0.990	0.733	0.863	0.999	1.375	0.487	1.222
168	1.544	1.446	1.073	1.246	2.439	3.591	1.436	1.501	0.728	1.024	0.759	0.895	1.025	1.430	0.511	1.271

Table I.5 Measured and predicted total creep coefficients and basic creep coefficients for different ages of loading for mix A1 concrete specimens

Duration of Load (Days)	Creep Coefficient ϕ (Creep Factor)																	
	Total Creep									Basic Creep								
	Measured	Predicted								Measured	Predicted							
		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	AS 3600 (1988)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	AS 3600 (1988)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)
1	0.390	0.069	0.[illegible]82	0.176	0.208	0.262	0.782	0.407	0.624	0.072	0.049	0.058	0.126	0.023	0.110	0.579	0.132	0.578
2	0.535	0.255	0.304	0.255	0.277	0.431	0.925	0.500	0.668	0.136	0.181	0.215	0.183	0.023	0.181	0.626	0.163	0.612
3	0.571	0.364	0.434	0.313	0.370	0.573	1.030	0.564	0.698	0.112	0.258	0.306	0.223	0.046	0.241	0.659	0.183	0.634
4	0.680	0.441	0.526	0.361	0.416	0.689	1.116	0.615	0.721	0.113	0.313	0.371	0.260	0.046	0.290	0.684	0.200	0.651
5	0.734	0.501	0.597	0.403	0.462	0.795	1.191	0.657	0.741	0.121	0.359	0.422	0.289	0.046	0.334	0.706	0.214	0.666
6	0.752	0.550	0.656	0.438	0.508	0.887	1.257	0.693	0.759	0.122	0.390	0.463	0.315	0.046	0.373	0.726	0.226	0.678
7	0.834	0.592	0.705	0.471	0.554	0.972	1.318	0.725	0.774	0.162	0.419	0.498	0.338	0.046	0.408	0.743	0.236	0.690
14	1.205	0.778	0.927	0.634	0.785	1.407	1.643	0.887	0.857	0.266	0.551	0.655	0.455	0.092	0.591	0.835	0.291	0.749
21	1.459	0.887	1.057	0.742	0.901	1.686	1.880	0.996	0.918	0.383	0.628	0.747	0.533	0.116	0.708	0.902	0.328	0.792
28	1.713	0.964	1.149	0.822	1.086	1.891	2.072	1.079	0.967	0.463	0.683	0.812	0.591	0.139	0.795	0.956	0.357	0.826
56	2.257	1.151	1.371	1.022	1.247	2.358	2.614	1.299	1.106	0.772	0.815	0.969	0.734	0.185	0.991	1.114	0.437	0.921
112	2.756	1.337	[illegible].393	1.217	1.525	2.747	3.233	1.534	1.270	1.151	0.947	1.125	0.875	0.277	1.154	1.308	0.532	1.032
140	2.874	1.397	1.665	1.277	1.548	2.853	3.433	1.610	1.326	1.373	0.990	1.176	0.917	0.277	1.199	1.375	0.566	1.070
168	3.010	1.446	1.723	1.323	1.571	2.97[illegible]	3.591	1.671	1.371	1.419	1.024	1.217	0.951	0.323	1.230	1.430	0.595	1.102

Table L6 Measured and predicted total creep coefficients and basic creep coefficients for different ages of loading for mix A2 concrete specimens

Duration of Load (Days)	Creep Coefficient φ (Creep Factor)															
	Total Creep								Basic Creep							
	Measured	Predicted							Measured	Predicted						
		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)		BS 8110 (1985)	BS 8110 M-dified (1985)	ACI 209 (1992)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)
1	0.291	0.069	0.082	0.157	0.218	0.782	0.328	0.550	0.153	0.049	0.058	0.113	0.092	0.579	0.106	0.513
2	0.384	0.255	0.304	0.227	0.359	0.925	0.403	0.593	0.216	0.181	0.215	0.163	0.151	0.626	0.131	0.547
3	0.390	0.364	0.434	0.280	0.477	1.030	0.455	0.623	0.178	0.258	0.306	0.201	0.201	0.659	0.148	0.571
4	0.471	0.441	0.526	0.323	0.574	1.116	0.495	0.648	0.198	0.313	0.371	0.232	0.241	0.684	0.161	0.591
5	0.508	0.501	0.597	0.359	0.663	1.191	0.529	0.669	0.198	0.355	0.422	0.258	0.279	0.706	0.172	0.608
6	0.539	0.550	0.656	0.392	0.739	1.257	0.558	0.688	0.197	0.390	0.463	0.281	0.311	0.726	0.182	0.624
7	0.607	0.592	0.705	0.420	0.810	1.318	0.584	0.706	0.205	0.419	0.498	0.302	0.340	0.743	0.190	0.638
14	0.891	0.778	0.927	0.566	1.173	1.643	0.714	0.803	0.304	0.551	0.655	0.407	0.493	0.835	0.234	0.715
21	1.070	0.887	1.057	0.662	1.405	1.880	0.802	0.877	0.404	0.628	0.747	0.476	0.590	0.902	0.264	0.774
28	1.225	0.964	1.149	0.734	1.576	2.072	0.869	0.938	0.456	0.683	0.812	0.527	0.662	0.956	0.287	0.823
36	1.596	1.151	1.371	0.913	1.965	2.614	1.046	1.115	0.710	0.815	0.969	0.656	0.826	1.114	0.352	0.964
112	1.924	1.337	1.593	1.087	2.289	3.233	1.236	1.326	0.897	0.947	1.125	0.781	0.962	1.308	0.429	1.133
140	2.048	1.397	1.665	1.140	2.377	3.433	1.297	1.398	1.011	0.990	1.176	0.819	0.999	1.375	0.456	1.192
168	2.122	1.446	1.723	1.182	2.439	3.591	1.346	1.458	1.020	1.024	1.217	0.849	1.025	1.430	0.479	1.241

APPENDIX J

MEASURED AND PREDICTED SPECIFIC CREEP VALUES

Table J.1 Measured and predicted specific total creep and specific basic creep values for different ages of loading for mix Q1 concrete specimens

Duration of Load (Days)	Specific Creep																	
	Specific Total Creep (x10E-6/MPa)									Specific Basic Creep (x10E-6/MPa)								
	Measured	Predicted								Measured	Predicted							
		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	AS 3600 (1988)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	AS 3600 (1988)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)
1	6.620	2.160	2.052	6.287	10.010	8.032	26.490	14.907	26.096	9.234	1.530	1.458	4.518	1.112	3.375	19.639	4.840	24.141
2	12.036	8.020	7.619	9.103	13.347	13.241	31.321	18.336	27.920	7.937	5.681	5.413	6.541	1.112	5.564	21.206	5.957	25.527
3	13.841	11.447	10.875	11.204	17.796	17.583	34.884	20.689	29.131	[illegible]	8.108	7.727	8.050	2.225	7.388	22.309	6.726	26.424
4	18.957	13.879	13.185	12.920	20.021	21.164	37.812	22.533	30.077	0.721	9.831	9.368	9.284	2.225	8.892	23.188	7.331	27.114
5	19.258	15.765	14.977	14.386	22.245	24.420	40.344	24.071	30.871	2.432	11.167	10.642	10.337	2.225	10.261	23.933	7.837	27.686
6	[illegible]	17.307	16.441	15.672	24.470	27.242	42.599	25.401	31.565	2.941	12.259	11.682	11.261	2.225	11.446	24.587	8.276	28.182
7	[illegible]	18.610	17.679	16.821	26.695	29.847	44.646	26.580	32.187	2.245	13.182	12.562	12.087	2.225	12.541	25.174	8.666	28.624
14	34.604	24.469	23.246	22.655	37.817	43.197	55.670	32.520	35.466	2.533	17.332	16.517	16.279	4.449	18.150	28.287	10.654	30.913
21	42.427	27.897	26.502	26.503	43.379	51.771	63.704	36.502	37.826	9.243	19.760	18.830	19.044	5.561	21.752	30.545	12.016	32.524
28	45.737	30.329	28.812	29.375	52.277	58.066	70.196	39.555	39.729	10.128	21.483	20.472	21.108	6.674	24.397	32.385	13.081	33.801
56	62.286	36.188	34.379	36.524	60.063	72.392	88.538	47.598	45.125	20.022	25.633	24.427	26.245	8.898	30.417	37.743	16.017	37.334
112	77.332	42.047	39.945	43.509	73.410	84.331	109.534	56.244	51.473	26.541	29.784	28.382	31.264	13.347	35.433	44.316	19.511	41.408
140	81.845	43.934	41.737	45.630	74.522	87.587	116.314	59.030	53.621	32.888	31.120	29.655	32.788	13.347	36.801	46.578	20.754	42.803
168	86.359	45.475	43.201	47.297	75.635	89.866	121.660	61.267	55.378	33.218	32.211	30.696	33.986	15.572	37.759	48.428	21.810	43.965

Table J.2 Measured and predicted specific total creep and specific basic creep values for different ages of loading for mix Q2 concrete specimens

Duration of Load (Days)	Specific Creep															
	Specific Total Creep (x10E-6/MPa)								Specific Basic Creep (x10E-6/MPa)							
	Measured	Predicted							Measured	Predicted						
		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)
1	5.605	1.598	1.518	4.141	4.949	21.644	9.190	16.500	5.194	1.132	1.078	2.976	2.079	16.030	2.983	15.376
2	7.654	5.931	5.634	5.996	8.158	25.591	11.304	17.777	4.773	4.201	4.003	4.309	3.428	17.326	3.672	16.398
3	9.533	8.466	8.042	7.380	10.833	28.502	12.754	18.671	5.376	5.996	5.714	5.303	4.552	18.228	4.147	17.108
4	11.411	10.264	9.751	8.511	13.040	30.895	13.851	19.396	3.418	7.270	6.928	6.116	5.479	18.946	4.519	17.683
5	12.436	11.659	11.076	9.476	15.046	32.964	14.839	20.023	3.106	8.258	7.870	6.809	6.322	19.555	4.831	18.180
6	12.606	12.799	12.159	10.324	16.785	34.806	15.659	20.584	3.880	9.066	8.639	7.418	7.053	20.089	5.102	18.625
7	14.826	13.762	13.074	11.081	18.390	36.473	16.386	21.096	3.971	9.748	9.290	7.962	7.727	20.569	5.342	19.031
14	21.656	18.096	17.191	14.923	26.615	45.486	20.048	23.929	3.113	12.818	12.215	10.723	11.183	23.113	6.568	21.280
21	25.413	20.631	19.599	17.458	31.898	52.051	22.503	26.068	6.916	14.613	13.926	12.545	13.403	24.957	7.407	22.977
28	26.779	22.429	21.308	19.350	35.777	57.355	24.385	27.833	8.157	15.887	15.140	13.904	15.032	26.461	8.064	24.373
56	34.463	26.762	25.424	24.059	44.604	72.341	29.343	32.936	13.762	18.957	18.065	17.288	18.741	30.838	9.874	28.392
112	40.610	31.095	29.541	28.661	51.960	89.497	34.673	39.034	15.554	22.026	20.989	20.595	21.832	36.209	12.028	33.214
140	43.513	32.490	30.866	30.058	53.966	95.036	36.391	41.123	18.543	23.014	21.931	21.598	22.675	38.058	12.794	34.896
168	45.733	33.630	31.949	31.156	55.370	99.405	37.770	42.847	18.287	23.821	22.700	22.388	23.265	39.569	13.445	36.308

Table J.3 Measured and predicted specific total creep and specific basic creep values for different ages of loading for mix G1 concrete specimens

Duration of Load (Days)	Specific Creep																	
	Specific Total Creep (x10E-6/MPa)									Specific Basic Creep (x10E-6/MPa)								
	Measured	Predicted								Measured	Predicted							
		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	AS 3600 (1988)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	AS 3600 (1988)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)
1	15.317	2.489	1.846	6.631	9.496	7.909	26.202	14.547	25.804	5.471	1.763	1.307	4.765	1.055	3.323	19.406	4.723	23.874
2	18.862	9.240	6.853	9.600	12.661	13.039	30.981	17.893	27.611	4.197	6.545	4.851	6.898	1.055	5.478	20.975	5.813	25.248
3	21.226	13.189	9.782	11.816	15.882	17.314	34.505	20.189	28.814	4.400	9.342	6.924	8.491	2.110	7.275	22.067	6.564	26.140
4	23.363	15.991	11.860	13.626	18.992	20.841	37.402	21.989	29.753	7.558	11.327	8.395	9.791	2.110	8.757	22.937	7.154	26.826
5	25.934	18.164	13.472	15.172	21.102	24.047	39.906	23.490	30.542	4.807	12.866	9.536	10.902	2.110	10.104	23.673	7.648	27.396
6	29.204	19.940	14.789	16.529	23.212	26.826	42.137	24.788	31.232	4.420	14.124	10.469	11.877	2.110	11.271	24.320	8.076	27.891
7	30.681	21.442	15.902	17.741	25.322	29.391	44.162	25.938	31.851	4.032	15.188	11.257	12.743	2.110	12.349	24.901	8.457	28.332
14	39.841	28.193	20.910	23.893	35.873	42.536	55.066	31.735	35.118	7.376	19.970	14.801	17.169	4.220	17.872	27.980	10.397	30.619
21	45.455	32.142	23.838	27.952	41.149	50.980	63.012	35.621	37.472	13.079	22.767	16.874	20.085	5.275	21.420	30.213	11.726	32.232
28	50.773	34.944	25.917	30.981	49.590	57.178	69.434	38.600	39.372	16.607	24.752	18.345	22.262	6.331	24.025	32.033	12.765	33.514
56	61.115	41.695	30.924	38.521	56.975	71.286	87.576	46.449	44.760	23.958	29.534	21.890	27.679	8.441	29.952	37.333	15.630	37.062
112	73.820	48.446	35.931	45.888	69.637	83.042	108.345	54.887	51.100	36.079	34.316	25.434	32.973	12.661	34.892	43.835	19.040	41.138
140	78.252	50.619	37.543	48.125	70.692	86.249	115.051	57.606	53.245	40.244	35.855	26.575	34.581	12.661	36.239	46.072	20.253	42.562
168	80.616	52.395	38.860	49.883	71.747	88.493	120.340	59.788	55.000	41.265	37.113	27.507	35.844	14.771	37.182	47.902	21.283	43.732

Table J.4 Measured and predicted specific total creep and specific basic creep values for different ages of loading for mix G2 concrete specimens

Duration of Load (Days)	Specific Creep															
	Specific Total Creep (x10E-6/MPa)								Specific Basic Creep (x10E-6/MPa)							
	Measured	Predicted							Measured	Predicted						
		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)
1	[illegible].554	2.082	1.544	4.416	4.949	21.644	9.190	16.498	5.031	1.474	1.093	3.173	2.079	16.030	2.983	15.374
2	13.140	7.728	5.732	6.394	8.158	25.591	11.304	17.774	4.785	5.47[illegible]	4.037	4.594	3.428	17.326	[illegible].672	16.395
3	15.701	11.031	8.181	7.869	10.833	28.502	12.754	18.666	5.706	7.8[illegible]	[illegible]	5.655	4.552	18.22[illegible]	[illegible].147	17.103
4	16.726	13.374	9.919	9.075	13.040	30.895	13.891	19.390	7.178	9.4[illegible]	[illegible]	6.521	5.479	18.9[illegible]	[illegible].519	17.677
5	18.263	15.192	11.267	10.104	15.046	32.964	14.839	20.015	5.706	10.7[illegible]	[illegible]	7.261	6.32[illegible]	19.555	4.831	18.172
6	19.629	16.677	12.369	11.008	16.785	34.806	15.659	20.574	5.706	11.813	[illegible]	7.910	7.053	20.089	5.102	[illegible]
7	21.166	17.933	13.300	11.815	18.390	36.479	16.386	21.085	5.092	12.702	9.415	8.490	7.727	20.569	5.342	[illegible]
14	26.801	23.579	17.488	15.912	26.615	45.486	20.048	23.909	7.239	16.702	12.379	11.434	11.183	23.113	6.568	21.260
21	31.070	26.882	19.938	18.615	31.898	52.051	22.503	26.041	11.718	19.042	14.113	13.376	13.403	24.957	7.407	22.949
28	33.972	29.226	21.676	20.633	35.777	57.355	24.385	27.798	13.190	20.702	15.343	14.826	15.032	26.461	8.064	24.339
56	40.803	34.872	25.863	25.654	44.604	72.341	29.343	32.881	19.509	24.701	18.308	18.434	18.741	30.838	9.874	28.338
112	48.657	40.518	30.051	30.560	51.960	89.497	34.673	38.955	24.356	28.701	21.272	21.960	21.832	36.209	12.028	33.134
140	51.219	42.336	31.399	32.050	53.966	95.036	36.391	41.035	25.276	29.988	22.226	23.030	22.675	38.058	12.794	34.808
168	51.302	43.821	32.501	33.221	55.370	99.405	37.770	42.751	25.890	31.040	23.006	23.871	23.265	39.569	13.445	36.212

Table J.5 Measured and predicted specific total creep and specific basic creep values for different ages of loading for mix A1 concrete specimens

Duration of Load (Days)	Specific Creep																	
	Specific Total Creep (x10E-6/MPa)									Specific Basic Creep (x10E-6/MPa)								
	Measured	Predicted								Measured	Predicted							
		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	AS 3600 (1988)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	AS 3600 (1988)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)
1	9.965	1.780	2.121	4.973	5.878	7.098	24.377	12.044	21.245	2.088	1.261	1.498	3.537	0.653	2.982	18.054	3.910	19.670
2	13.676	6.607	7.873	7.200	7.837	11.702	28.822	14.814	22.756	3.954	4.680	5.561	5.122	0.653	4.917	19.514	4.813	20.828
3	14.604	9.431	11.238	8.862	10.449	15.538	32.101	16.715	23.770	3.269	6.680	7.937	6.304	1.306	6.529	20.530	5.434	21.567
4	17.388	11.434	13.626	10.220	11.755	18.704	34.796	18.205	24.168	3.281	8.099	9.624	7.270	1.306	7.859	21.338	5.923	22.177
5	18.779	12.988	15.477	11.379	13.061	21.581	37.126	19.447	25.241	3.524	9.200	10.932	8.094	1.306	9.068	22.024	6.332	22.670
6	19.243	14.258	16.990	12.397	14.367	24.075	39.201	20.522	25.832	3.535	10.099	12.000	8.818	1.306	10.116	22.625	6.686	23.101
7	21.331	15.331	18.270	13.306	15.673	26.377	41.085	21.474	26.363	4.706	10.860	12.904	9.465	1.306	11.083	23.166	7.001	23.486
14	30.840	20.158	24.022	17.920	22.204	38.175	51.229	26.273	29.194	7.747	14.279	16.967	12.747	2.612	16.040	26.031	8.608	25.511
21	37.335	22.982	27.387	20.964	25.469	45.752	58.622	29.491	31.232	11.147	16.279	19.343	14.912	3.265	19.224	28.108	9.708	26.959
28	43.829	24.986	29.775	23.236	30.694	51.315	64.596	31.957	32.920	13.471	17.698	21.030	16.528	3.918	21.561	29.802	10.568	28.118
56	57.746	29.8[illegible]	35.527	28.891	35.265	63.976	81.475	38.455	37.665	22.442	21.117	25.092	20.551	5.224	26.881	34.732	12.940	31.356
112	70.303	34.[illegible]	41.279	34.417	43.102	74.537	100.796	45.440	43.232	33.468	24.537	29.155	24.481	7.837	31.314	40.781	15.763	35.127
140	73.518	36.194	43.131	36.094	43.755	77.404	107.035	47.692	45.142	39.919	25.637	30.463	25.674	7.837	32.523	42.862	16.768	36.425
168	76.997	37.464	44.644	37.413	44.408	79.419	111.955	49.499	46.688	41.268	26.537	31.532	26.613	9.143	33.369	44.564	17.620	37.508

Table J.6 Measured and predicted specific total creep and specific basic creep values for different ages of loading for mix A2 concrete specimens

Duration of Load (Days)	Specific Creep															
	Specific Total Creep (x10E-6/MPa)								Specific Basic Creep (x10E-6/MPa)							
	Measured	Predicted							Measured	Predicted						
		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)		BS 8110 (1985)	BS 8110 Modified (1985)	ACI 209 (1992)	CEB-FIP (1970)	CEB-FIP (1978)	CEB-FIP (1990)	RILEM Model B3 (1995)
1	7.093	1.568	1.869	3.481	4.635	20.719	8.244	14.674	3.750	1.111	1.320	2.501	1.948	15.345	2.676	13.680
2	9.354	5.822	6.938	5.039	7.642	24.498	10.141	15.823	5.295	4.124	4.901	3.621	3.211	16.586	3.294	14.604
3	9.505	8.311	9.904	6.202	10.147	27.284	11.442	16.631	4.375	5.887	6.995	4.457	4.263	17.449	3.720	15.249
4	11.464	10.077	12.008	7.152	12.214	29.575	12.462	17.289	4.865	7.138	8.481	5.140	5.132	18.137	4.054	15.774
5	12.368	11.446	13.640	7.964	14.093	31.555	13.312	17.860	4.848	8.108	9.634	5.723	5.922	18.719	4.334	16.229
6	13.121	12.565	14.973	8.676	15.722	33.319	14.048	18.371	4.832	8.900	10.576	6.234	6.606	19.230	4.577	16.637
7	14.779	13.511	16.101	9.312	17.225	34.920	14.700	18.839	5.021	9.570	11.372	6.691	7.237	19.690	4.793	17.011
14	21.711	17.765	21.170	12.542	24.929	43.542	17.985	21.436	7.452	12.584	14.952	9.012	10.474	22.125	5.892	19.090
21	26.081	20.254	24.136	14.672	29.878	49.826	20.188	23.405	9.915	14.346	17.047	10.543	12.554	23.891	6.645	20.666
28	29.848	22.019	26.240	16.262	33.510	54.904	21.876	25.032	11.173	15.597	18.533	11.685	14.080	25.330	7.234	21.965
56	38.890	26.273	31.309	20.220	41.778	69.249	26.324	29.741	17.402	18.610	22.113	14.529	17.554	29.520	8.858	25.715
112	46.877	30.528	36.379	24.087	48.668	85.672	31.106	35.371	21.991	21.624	25.694	17.308	20.449	34.662	10.790	30.224
140	49.891	31.897	38.011	25.261	50.548	90.975	32.647	37.300	24.791	22.594	26.847	18.151	21.238	36.431	11.478	31.800
168	51.699	33.016	39.344	26.184	51.863	95.156	33.884	38.892	25.029	23.386	27.788	18.815	21.791	37.878	12.062	33.122

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