CREEP AND SHRINKAGE TESTS AND MODELING FOR THE KEAKALAH Stream BRIDGE

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and

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Acknowledgments

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Abstract

Research performed at the University of Hawaii proposed to determine the long-term creep and shrinkage behavior of concrete planned for use in the Kealakaha Stream Bridge on the island of Hawaii. The concrete used in this study was based on a 6000 psi mixture design proposed by Jas W. Glover Ltd., of Hilo, Hawaii. Jas W. Glover also provided the coarse and fine aggregates from their Hilo quarry for an accurate representation of the concrete intended for use in the bridge structure.

Concrete test specimens were mixed, cast and tested in the University of Hawaii Concrete Technology Laboratory. Two different water-cement ratios created two different concrete mixtures with slumps of 1.5 inch and 5.5 inch. The two mixtures intended to provide upper and lower performance bounds for the probable field concrete. Various concrete material properties were tested at concrete ages of 1, 3, 28 and 56 days. Standard ASTM creep and shrinkage tests were performed on concrete from the first mixture at 1, 3 and 28 days of age, and on the second mixture at 3 days of age. These tests will be maintained for at least one year. This report includes the data collected during the first 56 days after placing the first mixture, and 28 days after placing the second mixture.

To provide long-term predictions of the creep and shrinkage for fifty years, two analytical prediction models are utilized. These models are modified using the actual material properties and short-term creep and shrinkage test data with a simple best-fit procedure developed during a similar study on the H3 North Hālawa Valley Viaduct. The resulting modified analytical models are then extrapolated to fifty years. Recommendations are made regarding the long-term material parameters intended for use in the design of the Kealakaha Stream Bridge.
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Nomenclature

\( f'_c \)  
The mean 28-day concrete compressive strength in psi.

\( C(t) \)  
The specific creep.

\( C_d(t, t', t_o) \)  
The drying creep compliance function.

\( C_o(t, t') \)  
The basic creep compliance.

\( E_c \)  
The concrete modulus of elasticity.

\( E_o \)  
The concrete modulus of elasticity at the age of loading.

\( f'_c \)  
The concrete compressive strength.

\( f'_{cmte} \)  
The mean compressive strength of the concrete at the time of drying.

\( f_r \)  
The concrete modulus of rupture.

\( h \)  
The relative humidity expressed as a decimal.

\( J(t) \)  
The creep compliance function.

\( K \)  
The cement type.

\( n \)  
The number of measured data values.

\( p_1 \)  
The y-intercept of the regression line modification.

\( p_2 \)  
The slope of the regression line modification.

\( q_1 \)  
The instantaneous strain per unit stress.

\( t' \)  
The time at loading.

\( t_o \)  
The concrete age when curing ends.

\( v/s \)  
The volume to surface area ratio of the specimen.

\( w \)  
The unit weight of concrete.

\( \alpha \)  
The coefficient of thermal expansion.

\( \beta(h) \)  
The relative humidity function.
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta(t)$</td>
<td>Function of the time since drying and the specimen volume to surface area ratio.</td>
</tr>
<tr>
<td>$\varepsilon''(t)$</td>
<td>The modified Gardner model shrinkage prediction.</td>
</tr>
<tr>
<td>$\varepsilon'(t)$</td>
<td>The unmodified Gardner model shrinkage prediction.</td>
</tr>
<tr>
<td>$\varepsilon_c$</td>
<td>The inelastic creep strain at time $t$.</td>
</tr>
<tr>
<td>$\varepsilon_o$</td>
<td>The elastic strain at the time of loading, $t$.</td>
</tr>
<tr>
<td>$\varepsilon_t$</td>
<td>The total strain at time $t$.</td>
</tr>
<tr>
<td>$\varepsilon_{shu}$</td>
<td>The notional ultimate shrinkage strain.</td>
</tr>
<tr>
<td>$\varepsilon_{sh}$</td>
<td>The shrinkage strain at time $t$.</td>
</tr>
<tr>
<td>$\varepsilon_T$</td>
<td>The strain due to temperature at time $t$.</td>
</tr>
<tr>
<td>$\mu \varepsilon$</td>
<td>Unit for shrinkage, microstrain.</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Poisson's Ratio</td>
</tr>
<tr>
<td>$\phi''(t)$</td>
<td>The modified Bazant model creep coefficient prediction.</td>
</tr>
<tr>
<td>$\phi'(t)$</td>
<td>The unmodified Bazant model creep coefficient prediction.</td>
</tr>
<tr>
<td>$\phi(t)$</td>
<td>The creep coefficient.</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>The applied stress.</td>
</tr>
<tr>
<td>$\tau_{sh}$</td>
<td>The shrinkage half-time in days.</td>
</tr>
</tbody>
</table>
Chapter 1

Introduction

A new long-span box-girder bridge is planned for construction over the Kealakaha Stream on the island of Hawaii to replace the existing bridge. The project location is shown on a map of the island of Hawaii in Fig. 1.1. The new structure will span 680 feet, with the central span exceeding 360 feet. At the time of writing this report, the proposed structural system is a cast-in-place segmental post-tensioned cantilever box-girder as shown in Fig. 1.2. A high strength concrete mixture with an estimated 28-day compressive strength of 6000 psi will allow for early prestressing. These long spans are susceptible to excessive deflection and prestress losses if adequate design measures are not taken to predict the long-term behavior of the concrete. Creep and shrinkage are the most significant long-term factors. A recent study [7] found that current prediction methods underestimate creep and shrinkage by approximately 40%. This confirms prior research which found that these factors are more dramatic when Hawaiian aggregates are used as opposed to Mainland United States aggregates [9].

Structures designed without considering the effects of creep and shrinkage are susceptible to excessive deflections, loss of camber, prestress loss, and a redistribution of internal forces and moments. Deformations resulting from long-term behavior can be greater than the initial elastic deformations. Serviceability problems which arise include excessive deflection and cracking that may lead to corrosion of the reinforcing steel.

Creep within indeterminate structures can cause a critical redistribution of the internal forces. When creep occurs it induces strains in the concrete. Compatibility states that the strain in the reinforcing steel must be equal to that in the concrete. This equilibrium of strains means that even under service loads the reinforcing steel may be close to its yield stress. In the case of prestressed members, the prestress force in the tendons decreases as the member shortens due to creep and shrinkage. This can lead to cracking at the tensile fiber even though the member was designed to remain uncracked.

Shrinkage of reinforced concrete creates tension within the concrete which causes contraction, warping, and cracking. Shrinkage is often reduced by the
addition of reinforcing steel. Although, if the steel is not symmetric, bending and deflection may result.

![Diagram of Hawaii Island with project location marked.]

**Figure 1.1: Bridge location - Kealakaha Stream, Hawaii.**

The goal of this study is to determine the long-term behavior of the concrete to be used in the Kealakaha Stream Bridge Replacement Project on the island of Hawaii. The most reliable method for estimating long-term concrete behavior is performing ASTM creep and shrinkage tests on the structure’s design concrete mixture. The short-term laboratory test results can be compared with the current
analytical prediction models. The best model approximation can be modified using the empirical data and extrapolated for more accurate estimations of long-term creep and shrinkage behavior.

1.1 Present Study

In the present study the long-term behavior of concrete from the Hilo, Hawaii facility of Jas. W. Glover, LTD General Contractor is investigated for the Kealakaha Stream Bridge Replacement Project. This work is composed of several parts. The first part concerns the theory of creep and shrinkage. The mathematical formulation expressing total creep strain is presented in three current notations. The second part reviews the current analytical models for estimating the long-term behavior of concrete. Two models that most accurately predict creep and shrinkage of Hawaiian concrete are discussed. A technique is then presented that utilizes empirical data to modify the analytical model for specific design concrete mixtures.

Part three explains the work completed in the University of Hawaii Concrete Technology Laboratory. The experimental procedures, equipment, and testing are documented and shown to conform to the standards of the American Society of Testing and Materials. Part four presents the results of the laboratory data, model predictions, and the modified model predictions. Properties such as compressive strength, modulus of elasticity, Poisson’s ratio, tensile strength, creep and shrinkage are compared to published results. The last two parts discuss the findings and present final creep and shrinkage predictions for the Hawaiian concrete mixtures for use in the project.

![Figure 1.2: Proposed bridge configuration. (Elevation of structural system)](image)
Chapter 2

Theory

In general concrete is described as a nonhomogeneous, viscoelastic structural material with properties that change with age. This makes its behavior difficult to estimate. Under an applied load, concrete will initially deform elastically. Under a sustained load, time-dependent inelastic deformations occur. In addition, over time, concrete will lose volume and shrink as drying and curing occur. This time-dependent loss of volume is called shrinkage. The combination of the elastic deformation, inelastic deformation due to load, and shrinkage is called the creep. Creep strain, or plastic flow is the time-dependent deformation per unit length.

In general, creep refers to the total creep ($\varepsilon_t$). This includes shrinkage deformations. In this paper, creep will refer only to the inelastic deformation $\varepsilon_c$ and $\varepsilon_t$ will be referred to as the total strain. Many theories exist that attempt to explain the phenomenon of creep. It is beyond the scope of this paper to investigate the theories such as, Mechanical Deformation Theory, Crystalline Theory, Viscous Theory, Seepage Theory, etc. Although, an explanation of the concrete deformation components will be addressed. While shrinkage and creep are not independent phenomena, the superposition of the strains is assumed valid and each is considered independently.

2.1 Creep

Creep, or material flow, is the increase in strain with time due to a sustained load. The total strain at time $t$ can be written as the summation of the following components,

$$\varepsilon_t(t) = \varepsilon_o(t) + \varepsilon_c(t) + \varepsilon_{sh}(t) + \varepsilon_T(t)$$  \hspace{1cm} (2.1)

where $\varepsilon_o(t)$ is the elastic strain at the time of loading, $\varepsilon_c(t)$ is the inelastic creep strain at time $t$, $\varepsilon_{sh}(t)$ is the shrinkage strain at time $t$, and $\varepsilon_T(t)$ is the strain due to temperature at time $t$. In constant temperature conditions, $\varepsilon_T(t)$ is zero.
2.1.1 Elastic Strain

Elastic strain is the instantaneous deformation of a concrete specimen due to loading. Hooke’s law states that this deformation is linear and completely recoverable for stress levels less than 40% of the concrete compressive strength at the time of loading. The elastic strain is represented as,

$$\varepsilon_o(t) = \frac{\sigma}{E_o(t)}$$  \hspace{1cm} (2.2)

where $\sigma$ is the applied stress, and $E_o(t)$ is the concrete modulus of elasticity at the age of loading. Since $E_o(t)$ is time-dependent, so too is the elastic strain.

2.1.2 Creep Strain

Creep strain is the time-dependent inelastic deformation resulting from the applied load. Rearrangement of Eq.(2.1) yields the creep strain as,

$$\varepsilon_c(t) = \varepsilon(t) - \varepsilon_o(t) - \varepsilon_{sh}(t)$$  \hspace{1cm} (2.3)

at constant temperature conditions. The creep strain component is dependent on the factors mentioned in §2.3. Creep strain is partially recoverable if the applied load is reduced or removed. Typically it is divided into two components, basic creep and drying creep.

2.1.2.1 Basic Creep

Basic creep, an inelastic deformation, is defined as the time-dependent strain of a concrete specimen subjected to a constant applied stress with no change in moisture content [3]. This assumes the specimen is sealed to eliminate moisture loss or gain. This occurs in all concrete subjected to load, but is the primary component of creep in mass structures such as dams, and in underwater structures. The constituent composition of the concrete and the type of cement attribute the most to the amount of basic creep.

2.1.2.2 Drying Creep

Drying creep, an inelastic deformation, is the time-dependent increase in strain of a concrete specimen due to the reduction or loss of free water [3]. Water or moisture loss depends on the specimen volume/surface area ratio and the in situ environmental conditions.

2.1.3 Shrinkage Strain

The unloaded, unrestrained, constant temperature, time-dependent strain within a concrete specimen is termed shrinkage strain ($\varepsilon_{sh}(t)$). Basically, there are two types of shrinkage, plastic shrinkage and drying shrinkage [11].
Plastic shrinkage occurs during the first few hours after the concrete is poured into the forms. Exposed surfaces such as floor slabs, are more easily affected by exposure to dry air due to their large surface area. On the exposed-contact surface, water evaporates faster than it can be replaced by water from within the specimen. Drying shrinkage occurs after the concrete has attained its final set and the majority of the chemical hydration process is complete within the cement gel. It is the decrease in the specimen volume when it loses moisture by evaporation. Simply stated, it is the movement of water out of the gel structure of a concrete specimen. Shrinkage is an irreversible process. The shrinkage strain component is dependent on the factors mentioned in §2.3.

2.2 Creep Notation

There are three commonly used ways of mathematically expressing creep, namely, creep coefficient, specific creep, and the creep compliance function. The formulation of each is presented in the following sections.

2.2.1 Creep Coefficient

The creep coefficient for a specimen in a state of constant stress, is defined as the ratio of the creep strain ($\varepsilon_c(t)$) divided by the elastic strain ($\varepsilon_0(t)$) at time $t$. The dimensionless quantity is,

$$\phi(t) = \frac{\varepsilon_c(t)}{\varepsilon_0(t)}$$

Note that the equation for the elastic strain, Eq.(2.2), includes the time-dependent modulus of elasticity. Total strain represented using the creep coefficient is, given by,

$$\varepsilon_t(t) = \frac{\sigma}{E_0} \left( \frac{\phi(t) + 1}{E_0} \right) + \varepsilon_{sh}$$

2.2.2 Specific Creep

Specific creep, $C(t)$, is defined as the creep per unit stress. Specific creep is popular when comparing creep results since it does not involve the time-dependent elastic modulus or the different applied axial stresses. Algebraically it is represented by,

$$C(t) = \frac{\varepsilon_t(t)}{\sigma}$$

The total time-dependent creep strain using the specific creep function is,

$$\varepsilon_t(t) = \sigma \left( \frac{1}{E_0} + C(t) \right) + \varepsilon_{sh}$$
2.2.3 Creep Compliance Function

The creep compliance function \( J(t, t') \) is more comprehensive than the creep coefficient and the specific creep functions. It is a function of the elastic and inelastic creep strain at time \( t \) due to an applied unit axial stress at time \( t' \). It is written,

\[
J(t, t') = q_1 + C_0(t, t') + C_d(t, t', t_o)
\]  
(2.8)

where \( q_1 \) is the instantaneous strain per unit stress, \( C_0(t, t') \) is the basic creep compliance, \( C_d(t, t', t_o) \) is the drying creep compliance function, \( t' \) is the time at loading, and \( t_o \) is the concrete age when curing ends. In terms of the creep compliance function, the total creep strain is,

\[
\varepsilon_c(t) = J(t, t')\sigma + \varepsilon_{sh}(t)
\]  
(2.9)

2.3 Factors Affecting Creep and Shrinkage

Creep and shrinkage strains depend on the concrete constituents and the environment where casting, curing, and loading occur [12]. As a result of the high dependence on the environmental conditions, concrete from the same batch when exposed to different environments can have very different strains. Past research have shown many different factors affect creep and shrinkage, the primary factors are listed below.

2.3.1 Factors Affecting Creep

- Water-cement ratio - For a given cement content, concrete with a higher w/c ratio will exhibit greater creep than that with a lower w/c ratio.

- Cement - Creep is approximately proportional to the amount of cement paste. Type III and Type IV cements produce concrete that shows greater creep than concrete produced with Type I cement [11].

- Aggregate quantity - Increasing the aggregate quantity with respect to the total volume effectively reduces the paste content within the mixture, therefore reducing the creep.

- Aggregate properties - Stiffer aggregates or aggregates with a higher modulus of elasticity creep less than softer or lower modulus aggregates. Greater porosity also increases creep.

- Concrete age - Increasing the curing period increases the strength and modulus of elasticity which results in a decrease of creep.
• Relative humidity - One of the two components of creep strain is drying creep. If drying is reduced by increasing the relative humidity, this can significantly reduce the creep strain.

• Temperature - Increases in temperature at high stress levels may increase creep. Under normal loads and temperature, the effect is relatively minor.

2.3.2 Factors Affecting Shrinkage

• Water-cement ratio (w/c ratio) - Concrete with a high w/c ratio exhibits significantly greater shrinkage than that with a lower w/c ratio.

• Cement - Rapid-hardening cements produce concrete that shrinks more than concrete produced with other types of cement.

• Aggregate quantity - Aggregate acts to restrain the shrinkage of cement paste. Increasing the aggregate content decreases shrinkage. Shrinkage occurs in the cement paste, with less cement paste there is less shrinkage [11].

• Aggregate properties - Similar to the effect on creep, stiffer aggregates or aggregates with a higher modulus of elasticity shrink less than softer or lower modulus aggregates. Increasing porosity also increases shrinkage.

• Specimen size - Both the rate and total magnitude of shrinkage decrease with an increase in volume to surface area ratio of the concrete element.

• Relative humidity - The rate of shrinkage is lower at high states of relative humidity [11].

• Temperature - Shrinkage becomes stabilized at low temperatures [11].
Chapter 3

Analytical Models

Creep and shrinkage of concrete are known to follow a distinct pattern. Each occurs rapidly at the outset, either under applied stress or following curing. As time progresses, creep and shrinkage strains tend to approach a limiting value asymptotically. Although, there is much debate if they ever reach a finite limiting value.

Numerous analytical models predicting creep and shrinkage have been developed over the years with fair results. They can be divided into two classes, or theories. The first assumes a limiting value for both creep and shrinkage where the mathematical models use exponential and hyperbolic functions for the prediction. The second class assumes there is no limiting value and utilizes power and logarithmic functions to model creep and shrinkage behavior.

Although it is not known for certain which theory is correct, quite often it is irrelevant during the lifetime of the structure in question. In general, approximately 75% of creep is assumed within the first year, and roughly 90% of shrinkage occurs within one year [10]. It must be noted that many factors affect creep and shrinkage and these estimates can vary from one situation to the next.

Research performed at the University of Hawaii in 1997 evaluated predictions of a number of analytical models currently in use in the structural design community. Some models are based on empirical data (28-day $f' c$, $E_c$, w/c ratio, etc.) and others are exclusively analytical. The models tested were:

1. American Concrete Institute's 1992 analytical model for creep and shrinkage (ACI-209R-92) [1].

2. Comité Euro-International du Béton/Federation International de la Pre-contrainte - Europe's 1993 analytical model for creep and shrinkage [5].

3. Z. P. Bazant and Baweja's short form B3 analytical model for creep and shrinkage [4].

4. N. J. Gardner and Zhou's 1994 analytical model for creep and shrinkage [8].
The model predictions were compared to data obtained from the North Halawa Valley Viaduct Project. This viaduct is a cast-in-place post-tensioned box girder bridge located on Oahu, Hawaii. Concrete cylinders were taken from specific bridge sections and immediately sent to the private testing facility at Construction Technology Laboratories (CTL) in Skokie, Illinois. CTL completed material tests such as compressive strength ($f'_c$), modulus of elasticity ($E_c$), and coefficient of thermal expansion ($\alpha$). Cylinders were also tested for creep and shrinkage at different concrete ages. Test data were recorded up to 800 days. The data analysis was performed at the University of Hawaii [7].

All the models mentioned were compared to the Viaduct’s test data. Empirical data from the Viaduct project was used in each model as required. Comparisons between the models and the test data included “Goodness of Fit” testing. All of the models underestimated the creep and shrinkage by as much as 60%, although, some models fit the test data better than others. In order to improve the analytical predictions, the models were modified based on the first 28 days of empirical data. A simple linear regression technique was shown effective for most of the predictive models. Using this technique, it was determined that the Bazant Creep Model was the best for predicting creep of Hawaiian concrete, while the Gardner Shrinkage Model performed best at predicting shrinkage of Hawaiian concrete. This study of, Creep and Shrinkage Tests and Modeling for the Kealakaha Stream Bridge, will use the Bazant Creep Model and the Gardner Shrinkage Model based on short-term test data to predict the long-term behavior of the Hawaii Island concrete.

3.1 Bazant Creep Model

The creep model proposed by Bazant and Baweja is currently in consideration as a replacement to the current ACI 209R-92 creep prediction model. This model is based on a creep compliance function ($J(t)$), not the standard creep coefficient ($\phi(t)$). Mathematically, the creep compliance function is based on a double-log power expression. The justification of this revision by Bazant is that the model is simpler, and more accurate than the current ACI model. The model is intended for relatively simple structures. A complex form of the model exists for more detailed structures.

The creep compliance function is the sum of the instantaneous elastic deflection due to loading, the basic creep, and the drying creep. Algebraically it is represented as,

$$J(t, t') = q_1 + C_o(t, t') + C_d(t, t', t_o)$$

3.1

where $q_1$ is the instantaneous strain due to a unit stress, $C_o(t, t')$ is the basic creep compliance, $C_d(t, t', t_o)$ is the drying creep compliance, $t$ is the concrete age in days, $t'$ is the age at loading, and $t_o$ is the age at the start of drying. The
CHAPTER 3. ANALYTICAL MODELS

The instantaneous strain component \( q_1 \) representing the strain per unit stress is,

\[
q_1 = \frac{0.6 \times 10^6}{E_{28}}
\]  

(3.2)

where \( E_{28} \) is the modulus of elasticity at 28 days measured in psi. The basic creep compliance \( (C_o(t, t')) \) represents basic creep. This double-log function is denoted by,

\[
C_o(t, t') = q_o \ln \left[ 1 + 0.3 \left( \frac{1}{\sqrt{t'}} + 0.001 \right) (t - t')^{0.1} \right]
\]  

(3.3)

and

\[
q_o = \frac{200}{\sqrt{f'_c}}
\]  

(3.4)

where \( f'_c \) is the mean 28-day compressive strength in psi. The drying creep compliance \( C_d(t, t', t_o) \) depends on the concrete and environmental properties such as the compressive strength, relative humidity, and age. It is expressed as,

\[
C_d(t, t', t_o) = q_5 \sqrt{e^{-3H(t)} - e^{-3H(t')}} \quad \text{for } t' \geq t_o
\]  

(3.5)

and

\[
q_5 = \frac{6000}{f'_c}
\]  

(3.6)

\[
H(t) = 1 - (1 - h)S(t)
\]  

(3.7)

\[
S(t) = \tanh \sqrt{\frac{t - t_o}{\tau_{sh}}}
\]  

(3.8)

\[
\tau_{sh} = 128 \left( \frac{v}{s} \right)^2
\]  

(3.9)

where \( h \) is the relative humidity expressed as a decimal, \( \tau_{sh} \) is the shrinkage half-time in days, and \( v/s \) is the volume to surface area ratio of the specimen.

### 3.2 Gardner Shrinkage Model

Gardner and Zhou also proposed a shrinkage model to replace the current ACI 209R-92 models. The Gardner shrinkage model relates the time-dependent concrete strength and stiffness to the progression of shrinkage. Data required by the
model as input is information readily available to the practicing engineer. This has contributed to its popularity for shrinkage prediction. Special allowances such as different types of cement, admixtures, and aggregate strength can be accounted for in the predictions.

The model expresses shrinkage as a product of the notional ultimate shrinkage strain and the modification factors. These modification factors include the effects of relative humidity, drying time, and the volume to surface area ratio. Gardner’s equation for shrinkage strain at time \( t \) is written as,

\[
\varepsilon_{sh}(t) = \varepsilon_{shu} \beta(h) \beta(t)
\]  

(3.10)

where \( \beta(h) \) is the relative humidity function, and \( \beta(t) \) is a function that incorporates the time since drying and the specimen volume to surface area ratio. The notional ultimate shrinkage strain \( \varepsilon_{shu} \) accounts for the cement type and the concrete strength at the time of drying. Expressed in microstrain \( (\mu \varepsilon) \),

\[
\varepsilon_{shu} = 857K \sqrt{\frac{4000}{f'_{cmtc}}} \times 10^{-6}
\]  

(3.11)

where \( f'_{cmtc} \) is the mean compressive strength of the concrete at the time of drying, and

\[
K = \begin{cases} 
1.00 & \text{For Type I Cement.} \\
0.70 & \text{For Type II Cement.} \\
1.33 & \text{For Type III Cement.}
\end{cases}
\]

The relative humidity function \( (\beta(h)) \) considers the environmental effects on the shrinkage strain. The expression, where \( h \) is expressed as a decimal, is,

\[
\beta(h) = (1 - 1.18h^4)
\]  

(3.12)

In the case of a sealed specimen, \( \beta(h) \) is equal to zero. The shrinkage time function is

\[
\beta(t) = \left[ \frac{7.27 + \ln(t - t_c)}{17.18} \right] \left[ \frac{t - t_c}{t - t_c + 8.06(v/s)^2} \right]
\]  

(3.13)

where \( t - t_c \) is the number of days since drying began. The term \( \beta(t) \) uses Gardner’s creep time function, \( \phi(t) \), and a second term representing the time since drying began. The first term in the expression is the creep time function.

### 3.3 Modification of Bazant and Gardner Models

Predictive models attempt to calculate results for a wide range of concrete types and geographic regions. It would then be expected that there is some discrepancy
between the predicted and measured results. Since no rigorous theoretical model has been developed to date that simulates the long-term concrete behavior, the problem is handled using short-term empirical data. This includes creep and shrinkage tests performed on a specific concrete mixture. This compares short-term empirical data with model predictions and adjusts the model accordingly. Bazant [2] suggests a modification procedure described in §3.3.1.

### 3.3.1 Regression Analysis

The regression analysis procedure compares the short-term measured data with the model prediction. The values are plotted as shown in Fig. B.4. The model data ($\phi'(t)$) is the abscissa and the measured data ($\phi(t)$) is the ordinate. If there was exact agreement between $\phi'(t)$ and $\phi(t)$, there would be a straight line passing through the origin with a slope of 1. Rarely will this occur. A more realistic plot is shown in Fig. B.4. A least-squares regression is calculated between the model and test data resulting in a regression line (Fig. B.4). The model predictions can then be adjusted to modified creep coefficients ($\phi''(t)$) as follows,

$$\phi''(t) = p_2 \phi'(t) + p_1$$  \hspace{1cm} (3.14)

where $p_2$ is the slope of the regression line, and $p_1$ is the $y$-intercept. The coefficients $p_1$ and $p_2$ are calculated by,

$$p_1 = \bar{\phi}' - \bar{\phi}$$  \hspace{1cm} (3.15a)

$$p_2 = \frac{n \sum_{i=1}^{n} \phi_i \phi_i' - \sum_{i=1}^{n} \phi_i \sum_{i=1}^{n} \phi_i'}{n \sum_{i=1}^{n} \phi_i^2 - \left( \sum_{i=1}^{n} \phi_i \right)^2}$$  \hspace{1cm} (3.15b)

where $\bar{\phi}'$ and $\bar{\phi}$ are the average values of the predictive model and the short-term data, and $n$ is the number of measured data values.

This linear regression modification was applied to the Bazant creep model and the Gardner shrinkage model using short-term empirical data from the creep and shrinkage tests described in chapter 4. The resulting modified predictions are presented in chapter 5.
Chapter 4

Laboratory Program

This chapter describes experimental work performed in the University of Hawaii Concrete Technology Laboratory as part of this study. The nominal 28-day compressive strength of the concrete proposed for the Kealakaha Stream Bridge project was 6000 psi. Cylinders produced from the design concrete mixture were tested for compressive strength ($f'_c$), modulus of elasticity ($E_c$), Poisson's ratio ($\nu$), modulus of rupture ($f_r$), creep, and shrinkage. Tests were completed using design mixtures with two slightly different water contents, at concrete ages of 1 day, 3 days, 28 days, and 56 days.

4.1 Concrete Constituents

Jas. W. Glover, LTD. provided a proposed concrete mixture design for the Kealakaha Stream Bridge project. The coarse and fine aggregates used in the laboratory tests we obtained directly from the Jas. W. Glover Hilo quarry.

The coarse aggregate was #67 basalt from the Hilo quarry. The specific gravity was 2.81 with a moisture content of 2.6% and an absorption of 2.6%. The fine aggregate was #4 basalt from the Hilo quarry. It had a specific gravity of 2.85, a moisture content of 1.5% and an absorption of 1.5%.

The cement used in the study was Type I Portland Cement obtained in bulk from the Hawaiian Cement Corporation. This cement is ground finer than standard Type I, which produces early strengths that are higher than would be expected for a standard Type I cement.

The anticipated travel time of the concrete ready-mixture trucks required admixtures in the mixture design. Daratard-17 produced by Grace Concrete Products was used to retard the hydration process during the travel time. Daratard-17 is a ready-to-use aqueous solution of hydroxylated organic compounds. It retards the initial and final set of the concrete. At an amount of 3 fl. oz. per 100 lbs of cement, it will retard the setting time at 70°F by 2–3 hours. In addition to retarding hydration, Daratard-17 also provides water-reduction, typically 8–10%.
CHAPTER 4. LABORATORY PROGRAM

The mixture design's low water-cement ratio (0.43) required the inclusion of a water reducing admixture. WRDA-HA produced by Grace Concrete Products was used as the water reducing agent. WRDA-HA is a ready-to-use aqueous solution of lignosulfonates containing a catalyst promoting a more complete hydration of the cement. Amounts of WRDA-HA typically range from 3–10 fl. oz. per 100 lbs of cement depending on the project.

4.2 Concrete Mixture Design

The procedures used in the laboratory followed the American Society of Testing Materials (ASTM) Standards. All deviations are indicated. The complete mixture design supplied by Jas. W. Glover, LTD. for 1 cubic yard of 6000 psi concrete is shown in Figs. A.1 and A.2. It was determined that the first set of tests would require 12 ft³ of concrete. Due to the volume limitations of the laboratory concrete mixer (6 ft³) two batches with the same proportions were produced on December 19, 1997, labeled P1 and P2. The laboratory mixtures for the cylinders and slump tests were proportioned for 0.222 cubic yards (6 ft³). The water content for these mixtures was reduced to account for the moisture in the coarse aggregate. This resulted in a water-cement ratio of 0.40 and a relatively low slump concrete (Fig. A.1). A second mixture, labeled P3, was made on January 19, 1998 (6 ft³) to provide results from a mixture with a slump at the high end of the suppliers mixture design (3–6 inch). This mixture included the total mixing water called for in the mixture design, resulting in a water-cement ratio of 0.43 and a slump of 5.5 inches (Fig. A.2).

4.3 Specimens

Creep frames were loaded at concrete ages of 1 day, 3 days, and 28 days for mixtures P1 and P2, and one creep frame for mixture P3 at 3 days. Each creep frame required 2 cylinders and 2 companion shrinkage cylinders. Other tests performed on the specific days were compressive strength ($f'_c$), modulus of elasticity ($E_c$), Poisson's ratio ($\nu$), and drying. Material properties were also measured at 56 days for mixtures P1 and P2. Beam modulus of rupture ($f_r$) tests were also performed at 3, 28 and 56 days for mixtures P1 and P2, and at 28 days for mixture P3. A total of 62 cylinders and 8 beams were required for adequate testing.

4.4 Mixing Procedure

Concrete mixing followed the recommendations of ASTM C 192-94. The following procedure was undertaken:

1. The concrete mixer was prepared with a "butter batch."
2. The aggregates and cement were then thoroughly mixed for 5 minutes in the mixer.

3. Water was added to the dry mixture.

4. The concrete slurry was then allowed to remain undisturbed for approximately 5 minutes.

5. Finally, the concrete was mixed again for 5 minutes.

Slump tests conforming to the procedures of ASTM C 143-95 were immediately taken after final mixing. The results are listed in Figs. A.1 and A.2.

4.5 Concrete Specimens

All the cylinders prepared in the UH Concrete Technology Laboratory used 6×12 inch plastic cylindrical molds conforming to the provisions required by ASTM C 192. Prior to casting, the creep and shrinkage cylinder molds were fitted with gage points for strain measurements. A steel guide cylinder was placed over the mold to obtain three 8 inch gage lengths on vertical lines spaced 120° apart. The guide is a steel pipe with holes at the appropriate points. Six 1/4 inch diameter holes were drilled through the plastic molds. Brass hexagonal 1 inch studs were held inside the mold by screws on the outside. The molds were then ready for concrete.

Casting each specimen involved filling the mold in increments of one-third its volume. Then as each third was added, the concrete was rodded 25 times with a 5/8-inch diameter steel rod such that it did not disturb the previous third volume of concrete. Rodding insured uniformity within the mold and prevented separation of the constituents.

After rodding, the specimens were tapped, trowelled off, and capped. Each cylinder was appropriately marked and covered with wet burlap, plastic and placed under lights to simulate the site conditions during curing. The cylinders remained in their molds at 100% humidity until testing.

The 6×6×24 inch beams were cast into steel and wood forms coated with form release. Casting followed the identical procedure as the cylinders. These forms had no caps so a plastic sheet served to contain the moisture. They were dried and cured alongside the cylinders.

4.6 Specimen Preparation for Testing

On the day of testing, the cylinders were removed from their molds, weighed, and measured (length and diameter). The results are listed in Figs. A.3–A.9. Following the provisions of ASTM C 617-94, each cylinder was capped with a sulfur
compound that ensured a plane bearing surface for testing. The sulfur compound is commercially known as CylCap and is produced by Thermal Ceramics.

After capping, locator screws were fitted to the creep and shrinkage cylinders. These are 1/4 inch diameter stainless steel screws anchored to the brass studs embedded in the concrete. Each screw has a small drill hole in the head to locate one end of a caliper strain gage. All the screws and studs were bonded to the concrete with a 2-part epoxy to prevent movement due to possible cracking in the concrete around the studs.

4.7 Strain Gages

Creep and shrinkage strains were measured mechanically using the locator points installed on the periphery of the test specimens. Each cylinder had three sets of two points for measurement. The gage length on both the creep and shrinkage cylinders was 8 inch. A portable caliper strain gage measured the relative distance between the gage points attached to the concrete. The device used in this study was a Soiltest Model C21 Strain Gage with a resolution of 0.0001 inch. It is similar to a precision caliper except its locating points are conical. To account for any temperature effects the device was referenced to an invar bar prior to each set of measurements. The readings were adjusted by the different reference length in the event of temperature changes. The same instrument and reading technique was used throughout the study.

4.8 Materials Testing

All concrete testing followed the appropriate ASTM testing guidelines. For each of the three batches, 3 cylinders were required for testing compressive strength ($f'_c$), and at least 2 cylinders were used for modulus of elasticity ($E_c$) and Poisson’s Ratio ($\nu$) at 1, 3, 28, and 56 days. Modulus of rupture beam tests were conducted at 28 and 56 days. Results of these tests are given in Figs. A.3–A.10. Laboratory creep and shrinkage tests were initiated at 1, 3, and 28 days for batch P1 and P2, and at 3 days for batch P3.

Compressive strength testing followed the provisions of ASTM C 39-96. Cylinders were loaded until failure in a 300,000 lb. capacity Rehle Universal Testing Machine. The load reading on the machine was calibrated with a 700,000 lb. load cell manufactured by Construction Technology Laboratories (CTL). Agreement was within 1%. Load was applied constantly at 20–50 psi/sec depending on the anticipated failure. The compressive strength ($f'_c$) was determined by dividing the applied load by the measured cross-sectional.

Modulus of elasticity tests were performed in accordance with ASTM C 469-94. Strains were measured at 10% and 40% of $f'_c$. Load was applied constantly at 20–50 psi/sec. A compressometer with a least count of 0.0001 inch measured
CHAPTER 4. LABORATORY PROGRAM

axial and transverse concrete displacements. The displacement readings were then converted into strain. The difference in the applied stress divided by the difference in strain determined the modulus of elasticity ($E_c$).

Poisson's ratio was tested simultaneously with the modulus of elasticity tests. The testing was done in accordance with ASTM C 469-94. Poisson's ratio ($\nu$) was determined by the difference in the transverse strain at 10% and 40% $f'_c$ divided by the difference in the axial strain.

The tensile strength of the concrete was measured using the modulus of rupture tests following the guidelines of ASTM C 78-94. The 6×6×24 inch plain concrete beams with a span of 18 inch were subjected to a two point concentrated load located at third points of the beam. This ensured the center third of the beam span was in pure bending. The load at rupture was used to compute the moment, which in turn determined the stress ($f_r$) at the extreme tension fiber. Results are given in Fig. A.10.

4.9 Creep and Shrinkage Tests

Creep and shrinkage tests were performed in the UH Concrete Technology Laboratory in accordance with ASTM C 512-87. Each creep test consisted of a single creep frame containing two instrumented concrete cylinders. Companion shrinkage cylinders were stored alongside the creep frame. Fig. 4.1 shows a typical creep frame. The temperature in the laboratory varied from 21°C to 25°C with an average of 23°C. The relative humidity varied from 52% to 58% with an average of 55%.

4.9.1 Creep Frames

The creep frames consisted of a steel head plate bearing on the concrete specimens. Threaded rods and steel coil springs maintain the compressive load. The springs ensure that a long-term constant stress is applied. This configuration conformed to the guidelines set by ASTM C 512-87. Each frame contained four stacked cylinders, two test specimens in the center and dummy cylinders on the top and bottom to minimize nonuniform stresses between the steel plates and the cylinders.

Prior to loading, extreme care was taken to ensure the cylinders were concentrically aligned in the frame. Load was applied to the frames using a portable hydraulic jack and load cell. The frames were preloaded to 250 psi and the load uniformity was checked. Then loading was increased to 40% of $f'_c$. This load was periodically checked and adjusted if needed.

Shrinkage cylinders were stored (load-free) adjacent to the creep frames in the same stable environment room. They too were capped to ensure the same volume to surface area ratio as the creep cylinders.
4.10 Data Acquisition

Creep and shrinkage measurements followed those outlined in ASTM C 512-87. Readings were taken:

1. Immediately after stripping the molds. (Both creep and shrinkage cylinders)
2. Just prior to loading in the frame. (Both creep and shrinkage)
3. Immediately after reaching peak load, 40% $f'_c$. (Only creep cylinders)
4. Two to six hours after initial loading. (Both creep and shrinkage cylinders)
5. Daily for the first week after loading. (Both creep and shrinkage cylinders)
6. Weekly for the first month after loading. (Both creep and shrinkage cylinders)
7. Monthly thereafter. (Both creep and shrinkage cylinders)

The deformation of each cylinder is computed as the average strain from the three readings at 120° around the cylinder. Total deformation of the creep cylinder is averaged for both cylinders in the creep frame. The shrinkage deformation is the average of the two shrinkage cylinders. All of the data collected from the four creep frames and associated shrinkage cylinders is tabulated in Appendix A.
Figure 4.1: Typical creep frame.
Chapter 5

Results

The results of the testing performed in the UH Concrete Technology Laboratory are presented in this chapter. The discussion begins with the material properties \((f'_c, E_c, \nu)\) at specific concrete ages, the creep results, and finally the shrinkage results. Comparisons to published literature are made when appropriate.

5.1 Material Property Test Results

5.1.1 Compressive Strength

Concrete compressive strength \((f'_c)\) results are presented in Table 5.1. The average results from all the cylinders tested from batches P1, P2, and P3 at concrete ages of 1, 3, 28 and 56 days are presented. The raw data is shown in Figs. A.3–A.9. The P1 and P2 28-day tests were actually performed at a concrete age of 31 days.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>1 Day (psi)</th>
<th>3 Days (psi)</th>
<th>28 Days (psi)</th>
<th>56 Days (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1_avg</td>
<td>4339</td>
<td>6407</td>
<td>7930</td>
<td>8585</td>
</tr>
<tr>
<td>P2_avg</td>
<td>3868</td>
<td>6371</td>
<td>7907</td>
<td>9723</td>
</tr>
<tr>
<td>P3_avg</td>
<td>3610</td>
<td>4966</td>
<td>7364</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 5.1: Compressive strength test results \((f'_c)\).

The compressive strength results are consistent with expectations. For all three batches, the strength increased with age. P1 and P2 results are quite similar, as expected when using exactly the same design mixture. At each different concrete age, the strength of P3 is less than that of P1 and P2. This is attributed to the higher water-cement ratio of mixture P3. Cordon [6] states that an increase in the water-cement ratio corresponds to a decrease in compressive strength. At 1-day, in all three batches, the strength exceeds the typical prestressing design
strength ($f'_{cd}$) of 3500 psi. It is important to note the small volume (6×12 inch) of the test cylinders compared to the large volume of a bridge section. In mass structures, the hydration process produces significant heat which accelerates the curing process and thus the compressive strength [12].

5.1.2 Modulus of Elasticity

Table 5.2 shows the average results of all cylinders tested for modulus of elasticity ($E_c$). The raw data is presented in Figs. A.3–A.9. Again, the P1 and P2 28-day results were actually measured at a concrete age of 31 days. Also included is the modulus of elasticity calculated using the ACI formula for concrete at an age of 28-days,

$$E_c = 33 \ w^{1.5} \sqrt{f'_{c}}$$

where $w$ is the unit weight of the concrete (lb/ft³), $f'_{c}$ is the 28-day compressive strength in psi, and $E_c$ is in psi, but converted into ksi for comparison. This comparison is only applicable for the 28-day tests.

Concretes with strengths above 6000 psi are referred to as high-strength concretes. For such concrete, the ACI Committee 36 recommends the equation [10],

$$E_c = 40,000 \sqrt{f'_{c}} + 1.0 \times 10^6$$

The results for the modulus of elasticity follow the same trend as the compressive strength results. As the concrete age increases the modulus of elasticity increases. Increasing the water-cement ratio shows that the modulus of elasticity of P3 decreased with respect to the dryer mixtures P1 and P2. The 28-day results of the ACI predictive formula, Eq.(5.1), are quite different from the test results. Batches P1 and P3 have approximately 60% of the ACI estimated stiffness. Hamada et. al. [9] found a similar overestimation by the ACI Code when using Hawaiian aggregates.

The predictions using the newly adopted high-strength formula, Eq.(5.2) show better agreement. Batches P1 and P3 are approximately 75% of the predicted stiffness.

<table>
<thead>
<tr>
<th>Spec.</th>
<th>1 Day (ksi)</th>
<th>3 Day (ksi)</th>
<th>28 Day (ksi)</th>
<th>56 DAY (ksi)</th>
<th>ACI† (ksi)</th>
<th>Δ ACI† (%)</th>
<th>ACI‡ (ksi)</th>
<th>Δ ACI‡ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1avg</td>
<td>2727</td>
<td>2886</td>
<td>3526</td>
<td>3594</td>
<td>5701</td>
<td>61.8</td>
<td>4562</td>
<td>80.0</td>
</tr>
<tr>
<td>P2avg</td>
<td>2542</td>
<td>2860</td>
<td>3495</td>
<td>3697</td>
<td>5695</td>
<td>61.3</td>
<td>4557</td>
<td>77.0</td>
</tr>
<tr>
<td>P3avg</td>
<td>1986</td>
<td>2683</td>
<td>2979</td>
<td></td>
<td>5293</td>
<td>56.3</td>
<td>4432</td>
<td>67.0</td>
</tr>
</tbody>
</table>

† Eq.(5.1) ‡ Eq.(5.2)

Table 5.2: Modulus of elasticity test results ($E_c$).
5.1.3 Poisson’s Ratio

Poisson’s ratio test results are presented in Table 5.3. They too are the average results from all cylinders tested. The number of cylinders used in each test is indicated in the brackets for each batch. The raw data is in Figs. A.3–A.9.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>1 Day</th>
<th>3 Days</th>
<th>28 Days</th>
<th>56 Days</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1_{avg}</td>
<td>0.208 (1)</td>
<td>0.224 (1)</td>
<td>0.180 (1)</td>
<td>0.232 (1)</td>
<td>0.211 (4)</td>
</tr>
<tr>
<td>P2_{avg}</td>
<td>-</td>
<td>0.222 (1)</td>
<td>-</td>
<td>0.233 (1)</td>
<td>0.228 (2)</td>
</tr>
<tr>
<td>P3_{avg}</td>
<td>-</td>
<td>0.200 (3)</td>
<td>0.219 (2)</td>
<td>-</td>
<td>0.203 (5)</td>
</tr>
<tr>
<td>Average</td>
<td>0.208 (1)</td>
<td>0.204 (5)</td>
<td>0.206 (3)</td>
<td>0.233 (2)</td>
<td>0.210 (11)</td>
</tr>
</tbody>
</table>

Table 5.3: Poisson’s ratio test results (ν).

The apparatus used to test the Poisson ratio was rather intricate. It was not always possible to produce consistent results so some measured values were not recorded or were ignored. Since the ratio of axial strain to transverse strain is independent of concrete age and applied stress, the ratio should remain constant\(^1\). In order to get the most realistic value for Poisson’s ratio, all the tests were averaged, yielding a value of 0.210.

5.2 Tensile Strength

The results of the modulus of rupture beam tests are shown in Table 5.4. The results are presented in terms of the ratio of the modulus of rupture (\(f_r\)) to the square-root of the compressive strength, \(f_r/\sqrt{f_c}\). The raw data is in Fig. A.10. The ACI recommends using the following equation,

\[
f_r = 7.5 \sqrt{f_c}
\]

to determine the modulus of rupture for normal weight concrete.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>28 Days</th>
<th>56 Days</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1_{avg}</td>
<td>((f_r/\sqrt{f_c}))</td>
<td>((f_r/\sqrt{f_c}))</td>
</tr>
<tr>
<td>P2_{avg}</td>
<td>9.0</td>
<td>7.62</td>
</tr>
<tr>
<td>P3_{avg}</td>
<td>6.8</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 5.4: Modulus of rupture test results.

The test results appear consistent with this prediction, though the values for the P1 and P2 28-day tests are quite high. This is not unusual, given the

---

\(^1\)Assuming the applied stress is \(< 40% f_c\)
unpredictable nature of concrete tensile strength. The P1 and P2 56-day results and the P3 28-day results are roughly within the recommended range of 6.0-7.5.[11]

### 5.3 Creep Results

This section presents the results from the laboratory creep tests, the Bazant creep model, and the modified Bazant model using the initial laboratory data. Data is given for concrete loaded at ages of 1, 3, and 28 days at 40% $f'_c$. Values are based on a constant temperature of 23°C and relative humidity of 55%. Two mixtures are considered, batches P1 and P2 are referred to as mixture P1, while batch P3 is called mixture P3.

Based on Eq.(2.3), the creep strain at any time $t$ ($\epsilon_c(t)$) is determined from the total strain of a loaded cylinder ($\epsilon_t(t)$), less the elastic strain during loading ($\epsilon_o(t)$) and the shrinkage strain of an unloaded companion cylinder ($\epsilon_{sh}(t)$). Restating Eq.(2.3),

$$\epsilon_c(t) = \epsilon_t(t) - \epsilon_o(t) - \epsilon_{sh}(t)$$

Since each creep frame contains two loaded cylinders and two unloaded companion cylinders, creep is determined by averaging the readings using the following equation,

$$\bar{\epsilon}_{cr}(t) = \frac{1}{N} \sum_{i=1}^{N} [(\epsilon_t(t))_i - (\epsilon_o(t))_i] - \frac{1}{M} \sum_{j=1}^{M} (\epsilon_{sh}(t))_j$$  \hspace{1cm} (5.4)

The index $N$ denotes the six readings from the two creep cylinders, and $M$ refers to the six strain readings from the two companion shrinkage cylinders. For presentation purposes, creep strain was converted to creep coefficient ($\phi(t)$) and specific creep ($\tilde{C}(t)$) using Eq.(2.4) and Eq.(2.6), respectively.

The empirical data recorded from the four laboratory creep frames is shown in Figs. B.1 and B.2. Fig. B.1 shows the creep coefficients at time $t$ for mixtures P1 and P3 at the different concrete ages at loading. Fig. B.2 presents the creep per axial stress at time $t$ for mixture P1 and P3 at different ages of loading. Initially both the creep coefficient and the specific creep increase rapidly, then level out and approach a limiting value. Clearly, concrete creep responds inversely to the age at which it is loaded. Additionally, the higher water-cement ratio of mixture P3 increased the amount of creep compared to the drier mixture P1.

The complete description of the creep results for loading at different concrete ages follows in §5.3.1 - §5.3.4. Each data set, or age loaded, has its plots arranged in the Appendix B as:

1. The creep coefficient $\phi(t)$ for the measured data plotted with the Bazant model data at time $t$ for the initial data set (28 or 56 days).
2. Model creep coefficient $\phi'(t)$ at time $t$ versus the measured creep coefficient $\phi(t)$ for the initial data set (28 or 56 days). This shows the results of the linear regression procedure from §3.3.1.

3. Creep coefficient $\phi(t)$ for the measured data, predictions from the Bazant model, and the modified Bazant data at time $t$ projected for five years using the test data set (28 or 56 days).

4. Creep coefficient $\phi(t)$ for the measured data, predictions from the Bazant model, and the modified Bazant data at time $t$ projected for fifty years using the test data set (28 or 56 days).

Note, the specimens from mixture P1 loaded at ages of 1 and 3 days use 56 days of data in their model predictions. The P1 28-day and P3 3-day specimens use 28 days of data in the models. When referring to the plots, the number in parentheses in the legend denotes the amount of laboratory data used in the modified Bazant model.

5.3.1 Mixture P1 Loaded at 1-Day

Figs. B.3–B.7 show the creep data for mixture P1 loaded at 1-day. Fig. B.3 shows the measured data for the first 56 days ($\phi(t)$), and the Bazant model prediction for the same period ($\phi'(t)$). The creep specimens were stressed to 1684 psi. The Bazant model significantly overestimates the creep for 1-day loading. Fig. B.4 plots the relation between $\phi'(t)$ and $\phi(t)$ from Fig. B.3. The equation for the linear relationship between $\phi'(t)$ and $\phi(t)$ is $\phi(t) = 1.0995 \phi'(t) - 0.8069$, and is shown adjacent to the regression line (Fig. B.4). As described in §3.3.1, the slope ($p_2$) and the intercept ($p_1$) are used to modify the Bazant model predictions using Eq.(3.14). These modified predictions ($\phi'(t)$) are also plotted against the measured data in Fig. B.4. The linear regression through this data approaches the theoretically desired 1–1 relationship. The linear relationship becomes $\phi(t) = \phi'(t) - 3 \times 10^{-15}$.

Fig. B.5 shows the modified Bazant model compared to the measured results and the unmodified Bazant model prediction. The modification does extremely well fitting the model to the measured data. Fig. B.6 presents the measured data, predictions from the Bazant model, and the modified Bazant model projected five years using 56 days of measured data. At five years, the modified Bazant creep coefficient is approximately 20% less than that predicted by the unmodified Bazant model. Fig. B.7 plots the measured data, predictions from the Bazant model, and the modified Bazant model projected fifty years using 56 days of measured data. At fifty years, or 18,250 days, the modified Bazant creep coefficient is 15% less than that predicted by the unmodified Bazant model.
5.3.2 Mixture P1 Loaded at 3-Days

The 56 days of data and the regression analysis for P1 loaded at 3-days are shown in Figs. B.8 and B.9. Fig. B.10 plots the measured data, predictions from the Bazant model, and the modified Bazant data at time \( t \) projected five years using 56 days of measured data for mixture P1 loaded at 3 days. The creep specimens were stressed to 2571 psi. In this case, there is only a 1% difference between the five year predictions of the Bazant model and the modified Bazant model. Fig. B.11 plots the measured data, predictions from the Bazant model, and the modified Bazant data at time \( t \) projected fifty years using 56 days of measured data for mixture P1 loaded at 3 days. At fifty years or 18,250 days, the modified Bazant creep coefficient is 2.6% greater than that predicted by the unmodified Bazant model.

5.3.3 Mixture P1 Loaded at 28-Days

The 28 days of data and the regression analysis for P1 loaded at 28-days are shown in Figs. B.12 and B.13. Fig. B.14 presents the measured data, predictions from the Bazant model, and the modified Bazant data at time \( t \) projected five years using 28 days of measured data for mixture P1 loaded at 28 days. Creep specimens were stressed to 2600 psi. In this case, the Bazant model underestimates the modified Bazant model predictions by 46%. Fig. B.15 plots the measured data, predictions from the Bazant model, and the modified Bazant data at time \( t \) projected fifty years using 28 days of measured data for mixture P1 loaded at 28 days. At fifty years, or 18,250 days, the unmodified Bazant model underestimates the modified Bazant model, also by 46%.

5.3.4 Mixture P3 Loaded at 3-Days

The 28 days of data and the regression analysis for P3 loaded at 3-days are shown in Figs. B.16 and B.17. Fig. B.18 plots the measured data, predictions from the Bazant model, and the modified Bazant data at time \( t \) projected five years using 28 days of measured data for mixture P3 loaded at 3 days. This mixture has a higher water-cement ratio than mixture P1 (0.43 for P3 compared to 0.40 for P1). The creep specimens were stressed to 1980 psi. In this case, the Bazant model underestimates the modified Bazant model by 23%. Fig. B.19 plots the measured data, predictions from the Bazant model, and the modified Bazant data at time \( t \) projected fifty years using 28 days of measured data for mixture P3 loaded at 3 days. At fifty years, the unmodified Bazant model underestimates the modified Bazant model by approximately 26%.
5.3.5 Creep Results Summary

For comparison purposes, the five year modified Bazant model predictions for mixture P1 and P3 at the loading ages are plotted together in Figs. B.20 (creep coefficient) and B.21 (specific creep). Figs. B.22 and B.23 are the same as Figs. B.20 and B.21 but include estimates for mixture P3 at 1-day and 28-day loadings. These estimates are based on the assumption that mixture P3 will behave similarly to mixture P1. Since no laboratory data exists to verify these predictions, they should be used with caution.

Although concrete creep never reaches an asymptote, fifty years is considered the “ultimate” in this study. Fifty year modified Bazant model predictions for mixture P1 and P3 at the different loading ages are plotted together in Figs. B.24 (creep coefficient) and B.25 (specific creep). These plots include estimates for mixture P3 at 1-day and 28-day loadings. Again, since no laboratory data exists, these predictions should be used with caution.

Since the modulus of elasticity and applied stress are different for each concrete loading age, it is difficult to compare the creep coefficient plots. However, specific creep, or creep per axial stress, normalizes the data for easy comparisons of the long-term concrete behavior. When the age at loading increases, the specific creep decreases. Conversely, as the water-cement ratio increases so too does the creep. Table 5.5 summarizes the creep coefficient results measured in the laboratory, predicted by the Bazant model, and predicted by the modified Bazant model.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Age</th>
<th>Loaded</th>
<th>Data</th>
<th>Measured</th>
<th>Bazant</th>
<th>Mod. Bazant</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>days</td>
<td>days</td>
<td>days</td>
<td>$\phi(t)$</td>
<td>$C(t)$</td>
<td>$\phi'(t)$</td>
</tr>
<tr>
<td>P1-1</td>
<td>57</td>
<td>56</td>
<td>56</td>
<td>1.14</td>
<td>0.481</td>
<td>1.72</td>
</tr>
<tr>
<td>P1-3</td>
<td>59</td>
<td>56</td>
<td>56</td>
<td>1.18</td>
<td>0.409</td>
<td>1.40</td>
</tr>
<tr>
<td>P3-3</td>
<td>31</td>
<td>28</td>
<td>28</td>
<td>1.32</td>
<td>0.491</td>
<td>1.24</td>
</tr>
<tr>
<td>P1-28</td>
<td>56</td>
<td>28</td>
<td>28</td>
<td>0.98</td>
<td>0.280</td>
<td>0.60</td>
</tr>
</tbody>
</table>

Table 5.5: Measured and predicted creep coefficients at concrete ages of 1, 3, and 28 days.

5.4 Shrinkage Test Results

This section presents the results from the laboratory shrinkage tests, the Gardner shrinkage model, and the modified Gardner shrinkage model using the short-term laboratory data. Data is listed as drying ages of 1, 3, and 28 days. Values are based on a constant temperature of 23°C and relative humidity of 55%. Cylinders remained in their molds at 100% humidity until the appropriate age. Two mixtures are considered, batches P1 and P2 are called mixture P1, and batch P3 is called mixture P3. Shrinkage data from the shrinkage cylinders, adjacent to
CHAPTER 5. RESULTS

...the creep frames, is averaged using the equation,

\[ \bar{\varepsilon}_{sh}(t) = \frac{1}{M} \sum_{j=1}^{M} (\varepsilon_{sh}(t))_j \]  

(5.5)

where \( M \) is the summation over the six readings, three readings per cylinder. Shrinkage strain is presented in microstrain (\( \mu \varepsilon \)).

The empirical data recorded from the four sets of laboratory shrinkage cylinders is shown in Fig. C.1. This shows shrinkage strains at time \( t \) for mixture P1 and P3 at the different ages of drying. Initially the shrinkage occurs rapidly. However, as \( t \) increases, the shrinkage strains level out. The longer the concrete is exposed to drying, the lower the resulting shrinkage. The higher water-cement ratio of mixture P3 appears to results in slightly more shrinkage than the drier mixture P1.

A complete description of the shrinkage results for different concrete ages follows in §5.4.1 - §5.4.4. Figs. plotting each data set, or concrete age, are arranged in Appendix C as follows:

1. The shrinkage strain for the measured data plotted with the Gardner model data at time \( t \) (28 or 56 days).

2. Model shrinkage coefficient \( \varepsilon_{sh}(t) \) at time \( t \) versus the measured shrinkage-coefficient \( \varepsilon_{sh}(t) \) for the initial data set (28 or 56 days). This shows the result of the linear regression procedure from §3.3.1.

3. Shrinkage strain \( \varepsilon_{sh}(t) \) for the measured data, predictions from the Gardner model, and the modified Gardner model data at time \( t \) projected for five years using the test data set (28 or 56 days).

4. Shrinkage strain \( \varepsilon_{sh}(t) \) for the measured data, predictions from the Gardner model, and the modified Gardner model data at time \( t \) projected for fifty years using the test data set (28 or 56 days).

Note, the specimens from mixture P1 at ages of 1 and 3 days use 56 days of data to modify the Gardner model predictions. The P1 28-day and P3 3-day specimens use 28 days of data to modify the Gardner model. When referring to the plots, the number in parentheses in the legend denotes the amount of laboratory data used in the modified Gardner model.

In most cases, the shrinkage data is not as smooth as the creep data. This is attributed to small changes in strain between readings. As \( \Delta \varepsilon \) approaches to zero, errors in the mechanical device reading the cylinders become more pronounced.

5.4.1 Mixture P1 at 1-Day

Figs. C.2–C.6 show the data for mixture P1 with drying initiated at 1-day. Fig. C.2 shows the measured data and the Gardner model prediction for the first
56 days. The Gardner model slightly underestimates the laboratory data. Fig. C.3 plots the relation between the Gardner prediction $\epsilon_{sh}(t)$, and the test data $\epsilon_{sh}(t)$, from Fig. C.2. The equation for the linear relationship between $\epsilon_{sh}(t)$ and $\epsilon_{sh}(t)$ is located adjacent to the line ($\epsilon(t) = 1.042 \epsilon'(t) + 37.07$). Based on the procedure from §3.3.1, the model prediction was modified using the slope ($p_2$) and the intercept ($p_1$) to modify the Gardner model predictions, using Eq.(3.14). The plot now approaches the theoretically desired 1-1 relationship, where $\epsilon(t) = 1.023 \epsilon''(t) - 8.46$.

Fig. C.4 shows the modified Gardner model compared to the measured results and the unmodified Gardner model prediction. The modification does well fitting the measured data. Fig. C.5 presents the measured data, predictions from the Gardner model, and the modified Gardner model data at time $t$ projected five years using 56 days of measured data. The modified shrinkage prediction is slightly larger than that from the unmodified Gardner model. Fig. C.6 plots the measured data, predictions from the Gardner model, and the modified Gardner model data at time $t$ projected fifty years using 56 days of measured data. The modified shrinkage prediction is slightly larger than that from the unmodified Gardner model, 947$\mu e$ compared to 1009$\mu e$.

### 5.4.2 Mixture P1 at 3-Days

The regression analysis and plots for P1 3-days are in Figs. C.7–C.10. Fig. C.9 plots the measured data, predictions from the Gardner model, and the modified Gardner model data at time $t$ projected five years using 56 days of measured data for mixture P1 with the start of drying at 3 days. In this case, the Gardner model underestimates the modified Gardner model by approximately 18% at five years. Fig. C.10 presents the measured data, predictions from the Gardner model, and the modified Gardner model data at time $t$ projected fifty years using 56 days of measured data for mixture P1 with the start of drying at 3 days. The Gardner model underestimates the modified Gardner model by 17% at fifty years.

### 5.4.3 Mixture P1 at 28-Days

The regression analysis and plots for P1 28-days are in Figs. C.11–C.14. Fig. C.13 presents the measured data, predictions from the Gardner model, and the modified Gardner model data at time $t$ projected five years using 28 days of measured data for mixture P1 with the start of drying at 28 days. In this case, the Gardner model underestimates the modified Gardner model predictions by 11%. Fig. C.14 plots the measured data, predictions from the Gardner model, and the modified Gardner model data at time $t$ projected fifty years using 28 days of measured data for mixture P1 with the start of drying at 28 days. The Gardner model underestimates the modified Gardner model by 10% at fifty years, 638$\mu e$ compared to 708$\mu e$. 
5.4.4 Mixture P3 at 3-Days

The regression analysis and other plots for P3 3-days are in Figs. C.15–C.18. Fig. C.17 plots the measured data, predictions from the Gardner model, and the modified Gardner model data at time \( t \) projected five years using 28 days of measured data for mixture P3 with the start of drying at 3 days. This mixture has a higher water-cement ratio. Again, the Gardner model underestimates the modified Gardner model, but by only 5%. Fig. C.18 presents the measured data, predictions from the Gardner model, and the modified Gardner model data at time \( t \) projected fifty years using 28 days of measured data for mixture P3 with the start of drying at 3 days. The Gardner model underestimates the modified Gardner model by 4% at fifty years, 869\( \mu \varepsilon \) compared to 903\( \mu \varepsilon \).

5.4.5 Shrinkage Results Summary

For comparison purposes, the five year modified Gardner model predictions for mixture P1 and P3 at all drying ages are plotted together in Fig. C.19. When the age at which drying occurs increases, the resulting shrinkage decreases. The higher water-cement ratio of mixture P3 does not appear to increase the shrinkage. This may change as more data is collected from the P3 test cylinders (currently only 28 days of data). As a consequence, no estimates are made for mixture P3 at 1 day and 28 day drying ages. Table 5.6 summarizes the shrinkage strain results measured in the laboratory, predicted by the Gardner model, and predicted by the modified Gardner model. The unmodified Gardner model predictions were only slightly lower than the test data collected to date. Using the modified model to predict the behavior after five years, the shrinkage is between 5% and 17% higher than predicted by the unmodified model.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Age days</th>
<th>Drying days</th>
<th>Measured ( \mu \varepsilon )</th>
<th>Gardner ( \mu \varepsilon )</th>
<th>Modified Gardner ( \mu \varepsilon )</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1-1</td>
<td>57</td>
<td>56</td>
<td>523</td>
<td>473</td>
<td>527</td>
</tr>
<tr>
<td>P1-3</td>
<td>59</td>
<td>56</td>
<td>471</td>
<td>378</td>
<td>465</td>
</tr>
<tr>
<td>P3-3</td>
<td>31</td>
<td>28</td>
<td>421</td>
<td>329</td>
<td>385</td>
</tr>
<tr>
<td>P1-28</td>
<td>56</td>
<td>28</td>
<td>327</td>
<td>241</td>
<td>309</td>
</tr>
</tbody>
</table>

Table 5.6: Measured and predicted shrinkage at concrete age of 1, 3, and 28 days.

5.5 Comparison of Results

In this section, comparisons are made between the creep predictions by the modified Bazant model and shrinkage predictions from the modified Gardner model, with published literature. Since creep and shrinkage of concrete is dependent on
variations in aggregate, cement, environmental factors, etc., and the aggregate used is typically indigenous to a region, it is clear that creep and shrinkage will vary geographically. Therefore, it is necessary to test for creep and shrinkage regionally. Comparisons of concrete behavior are made with two studies using Hawaiian concrete and one study using Mainland United States concrete. Creep results are compared in specific creep \( C(t) \) and creep coefficient \( \phi(t) \), and shrinkage results use microstrain \( \mu \).

5.5.1 Durbin

M. P. Durbin of the University of Hawaii studied the long-term behavior of Hawaiian concrete used on the H3 North Halawa Valley Viaduct [7]. The aggregate was quarried locally at Hawaiian Cement’s Halawa quarry. Cylinders were cast from specific sections of the bridge and tested by CTL in Skokie, Illinois for creep and shrinkage according to ASTM. The values presented here are measured creep and shrinkage at one year. Table 5.7 compares the Durbin creep results for concrete loaded at 3 and 28 days at 40% \( f'_c \) with the modified Bazant model prediction at one year. The following discussion refers to specific creep since it normalizes the data with respect to the applied stress for easier comparison.

<table>
<thead>
<tr>
<th>Study</th>
<th>Age at Loading</th>
<th>Days Loaded</th>
<th>Spec. Crp</th>
<th>Crp Coef</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>days</td>
<td>days</td>
<td>( C(t) )</td>
<td>( \phi(t) )</td>
</tr>
<tr>
<td>Durbin Measured</td>
<td>3</td>
<td>365</td>
<td>1.15</td>
<td>3.65</td>
</tr>
<tr>
<td>Mod. Bazant P1-3</td>
<td>3</td>
<td>365</td>
<td>0.63</td>
<td>1.82</td>
</tr>
<tr>
<td>Mod. Bazant P3-3</td>
<td>3</td>
<td>365</td>
<td>0.89</td>
<td>2.38</td>
</tr>
<tr>
<td>Durbin Measured</td>
<td>28</td>
<td>365</td>
<td>0.92</td>
<td>3.32</td>
</tr>
<tr>
<td>Mod. Bazant P1-28</td>
<td>28</td>
<td>365</td>
<td>0.61</td>
<td>2.11</td>
</tr>
</tbody>
</table>

Table 5.7: Comparison between the measured Durbin [7] results and those predicted by the modified Bazant model.

Durbin’s results for the Halawa concrete show somewhat higher specific creep values than those predicted (for the Hilo concrete mixture) using the modified Bazant model. This is probably attributed to the differences between the Halawa quarry aggregate and that from the Hilo quarry.

The shrinkage results comparison is presented in Table 5.8. Durbin did not take data for drying cylinders at 28-days. Durbin’s results are significantly higher than those predicted for the Hilo concrete. Again, the difference is attributed to the different aggregates.

5.5.2 Hamada et. al.

In 1970, Hamada [9] investigated the behavior of Hawaiian aggregates under axial load and drying. Aggregate used in the study was 3/4 inch Halawa basalt from
Table 5.8: Comparison between the measured Durbin [7] shrinkage results and those predicted by the modified Gardner model.

<table>
<thead>
<tr>
<th>Study</th>
<th>Age at Drying</th>
<th>Days Drying</th>
<th>Shrinkage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Durbin Measured</td>
<td>3 days</td>
<td>365 days</td>
<td>1045 με</td>
</tr>
<tr>
<td>Mod. Gardner P1-3</td>
<td>3 days</td>
<td>365 days</td>
<td>675 με</td>
</tr>
<tr>
<td>Mod. Gardner P3-3</td>
<td>3 days</td>
<td>365 days</td>
<td>682 με</td>
</tr>
</tbody>
</table>

the Halawa quarry. A water reducer was incorporated in the design mixture. The results presented by Hamada are the predicted ultimate creep coefficient. Results from the modified Bazant model are the creep coefficients at fifty years loading (18,250 days). Table 5.9 compares the Hamada results for concrete loaded at 3 and 28 days at 40% $f'_c$ with the modified Bazant model. At 3 days loading age, the predictions from the modified Bazant model for the two mixtures P1 and P3 are greater than the results from Hamada. The 3-day P1 and P3 modified Bazant model predictions are 8% and 29% greater than Hamada’s results. For the 28 day tests, the modified Bazant model prediction is 37% higher than the Hamada result.

Table 5.9: Comparison between the Hamada [9] creep results and those predicted by the modified Bazant model.

<table>
<thead>
<tr>
<th>Study</th>
<th>Age at Loading</th>
<th>Days Loaded</th>
<th>Creep Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hamada Measured</td>
<td>3 days</td>
<td>∞</td>
<td>2.61 $\phi(t)$</td>
</tr>
<tr>
<td>Modified Bazant P1-3</td>
<td>3 days</td>
<td>18250 days</td>
<td>2.84</td>
</tr>
<tr>
<td>Modified Bazant P3-3</td>
<td>3 days</td>
<td>18250 days</td>
<td>3.70</td>
</tr>
<tr>
<td>Hamada Measured</td>
<td>28 days</td>
<td>∞</td>
<td>1.97</td>
</tr>
<tr>
<td>Modified Bazant P1-28</td>
<td>28 days</td>
<td>18250 days</td>
<td>3.12</td>
</tr>
</tbody>
</table>

The shrinkage results comparison is in Table 5.10. Hamada recorded data for drying cylinders only at 28-days. Hamada reports 19% more shrinkage than the modified Gardner prediction, 708 με compared to 877 με.

Table 5.10: Comparison between the Hamada [7] shrinkage results and that predicted by the modified Gardner model.

<table>
<thead>
<tr>
<th>Study</th>
<th>Age at Drying</th>
<th>Days Drying</th>
<th>Shrinkage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hamada Measured</td>
<td>28 days</td>
<td>∞</td>
<td>877 με</td>
</tr>
<tr>
<td>Mod. Gardner P1-28</td>
<td>28 days</td>
<td>18250 days</td>
<td>708 με</td>
</tr>
</tbody>
</table>
5.5.3 ACI 209 Committee

For comparison between Hawaiian and Mainland concrete, the American Concrete Institute Committee 209 [2] ultimate creep and shrinkage values are listed in Tables 5.11 and 5.12. The ACI data is based on work done by Meyers, Branson, and others. Table 5.11 compares the ACI 209 suggested ultimate creep coefficients with the Bazant model and the modified Bazant model predictions at fifty years.

<table>
<thead>
<tr>
<th>Study</th>
<th>Age at Loading</th>
<th>Days Loaded</th>
<th>Creep Coefficient φ(t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 209-92</td>
<td>∞</td>
<td>∞</td>
<td>2.35</td>
</tr>
<tr>
<td>Bazant P1-3</td>
<td>3</td>
<td>18250</td>
<td>2.76</td>
</tr>
<tr>
<td>Bazant P3-3</td>
<td>3</td>
<td>18250</td>
<td>2.74</td>
</tr>
<tr>
<td>Modified Bazant P1-3</td>
<td>3</td>
<td>18250</td>
<td>2.84</td>
</tr>
<tr>
<td>Modified Bazant P3-3</td>
<td>3</td>
<td>18250</td>
<td>3.70</td>
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<td>18250</td>
<td>1.67</td>
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<tr>
<td>Modified Bazant P1-28</td>
<td>28</td>
<td>18250</td>
<td>3.12</td>
</tr>
</tbody>
</table>

Table 5.11: Comparison between the ACI 209 [2] creep results and those predicted by the Bazant model and the modified Bazant model.

The ACI value agrees fairly well with the results from the Bazant model, although ACI is slightly lower. This is expected since the model is based on Mainland and European concretes. However, the modified Bazant model predictions for Hawaiian concrete are larger than the ACI 209 value.

A comparison of the ACI shrinkage results to the Garner predictions is given in Table 5.12. The ACI value represents the suggested ultimate shrinkage strain. The ACI shrinkage is greater than the Gardner and modified Gardner model predictions by 18% and 9% respectively.

<table>
<thead>
<tr>
<th>Study</th>
<th>Age at Drying</th>
<th>Days Drying</th>
<th>Shrinkage με</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 209-92</td>
<td>28</td>
<td>∞</td>
<td>780</td>
</tr>
<tr>
<td>Gardner P1-28</td>
<td>28</td>
<td>18250</td>
<td>638</td>
</tr>
<tr>
<td>Modified Gardner P1-28</td>
<td>28</td>
<td>18250</td>
<td>708</td>
</tr>
</tbody>
</table>

Table 5.12: Comparison between the ACI 209 [2] shrinkage results and those predicted by the Gardner model and the modified Gardner model.
Chapter 6

Conclusions and Recommendations

6.1 Conclusions

Research performed at the University of Hawaii determined the long-term creep and shrinkage behavior of concrete planned for use in the Kealakaha Stream Bridge on the island of Hawaii. The concrete used in this study was based on a 6000 psi mixture design proposed by Jas W. Glover Ltd., of Hilo, Hawaii. Jas W. Glover also provided both coarse and fine aggregates from their Hilo quarry for an accurate representation of the concrete intended for use in the bridge structure.

Concrete test specimens were mixed, cast and tested in the University of Hawaii Concrete Technology Laboratory. Two different water-cement ratios created two different concrete mixtures with slumps of 1.5 inches and 5.5 inches. The intent of the two mixtures was to provide upper and lower performance bounds for the probable field concrete mixture. Various concrete material property tests were performed at 1, 3, 28 and 56 days concrete age. Standard ASTM creep and shrinkage tests were performed on concrete from the first mixture starting at 1, 3, and 28 days of age, and on the second mixture starting at 3 days of age.

In order to provide long-term predictions of the creep and shrinkage for fifty years, two analytical prediction models are utilized. These models are modified using the actual material properties and the short-term creep and shrinkage test data using a simple best-fit procedure. The resulting modified semi-analytical models are extrapolated to fifty years yielding more accurate estimations regarding the long-term behavior of the design mixture for the Kealakaha Stream Bridge.

Observations generated from the analysis of the Hawaiian concrete are:

- Both concrete mixtures (1.5 inch and 5.5 inch slump) produced 1-day compressive strengths in excess of 3500 psi.

- Compressive strengths at 28-days were 7922 psi and 7364 psi for the two mixtures.
CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

- Results from the modulus of elasticity tests demonstrated that the stiffness of the Hawaiian concrete was substantially lower than that expected from the ACI formula given in Eq. (5.1). \(E_c = 57000 \sqrt{f'_c}\) for normal-weight concrete).

Better estimates were determined using the high-strength concrete formula Eq. (5.2). \(E_c = 40,000 \sqrt{f'_c} + 1.0 \times 10^6\). Though, even with this high-strength formula the Hawaiian concrete was approximately 75% the stiffness estimated estimated by the ACI.

- The average Poisson’s ratio of 0.210 is at the upper end of the accepted range for normal weight concrete (0.110–0.210 [10]).

- Current American Concrete Institute (ACI) design methods significantly underestimate creep and shrinkage of Hawaiian concrete.

- Creep and shrinkage predictions from the modified analytical models fall within the range of results from previously published research on Hawaiian concrete.

6.2 Recommendations

Based on the findings of this study, the following recommendations are furnished for use on the Kealakaha Stream Bridge Project.

- The authors stress exercising caution when using these design recommendations for creep and shrinkage. The models used are based on theories which do not exactly predict concrete behavior. Their intention is to provide a scientific estimation of the design mixture behavior. Even if the models predicted concrete behavior exactly, the variability between concrete mixtures is enough to constitute a range of values. Based on observations made during the H3 North Halawa Valley Viaduct study, the 95% confidence limits for creep with loading at 3-days and 28-days were 21% and 25% above and below the mean. The shrinkage 95% confidence limits for 3-day age at drying were 17% above and below the mean.

Applying the same limits to the mean obtained from averaging P1 and P3 mixture predictions establishes 95% confidence limits at approximately 10% to 15% above the P3 mixture prediction and 10% to 15% below the P1 predictions. The use of two concrete mixtures, one wet and one dry, was intended to estimate the range of probable concrete behavior. For design purposes, it is recommended that an additional 10% above and below these predictions be considered to ensure adequate structural performance at both extremes of creep and shrinkage behavior.
CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

- Table 6.1 presents the ultimate values for creep and shrinkage at fifty years of loading or drying. These values include different start days and two different design mixtures. Mixture P1 is the drier mixture (water-cement 0.40, slump 1.5 inch), and P3 is the wetter mixture (water-cement 0.43, slump 5.5 inch). All the values presented assume the 80% relative humidity conditions of the bridge location.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
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<td></td>
<td></td>
<td>Age</td>
<td>Creep Coef.</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>days</td>
<td>Age</td>
<td>$\phi(t)$</td>
<td>$C(t)$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>days</td>
<td></td>
<td>in$^2$/lb $\times 10^{-6}$</td>
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<tr>
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<td>18250</td>
<td>3.13†</td>
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<td>0.63</td>
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<td>18250</td>
<td>2.91†</td>
<td>0.98†</td>
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</tbody>
</table>

† Estimates based on P1 tests. ‡ Same as P1 predictions based on 3-day results

Table 6.1: Predicted values for creep and shrinkage at 50 years. Based on 6"Ø×12" cylinders at 80% relative humidity.

- Due to the inherent variability of the concrete behavior, the recommended design values and material variability were used in constructing upper and lower behavioral bounds. These are listed in Table 6.2. Figs. 6.1-6.7 plot creep coefficient, specific creep, and shrinkage for P1, P3, the mean of P1 and P3, and the upper and lower bounds.

<table>
<thead>
<tr>
<th>Loaded/Dried</th>
<th>Concrete</th>
<th>Creep Coef.</th>
<th>Specific Creep</th>
<th>Shrinkage</th>
</tr>
</thead>
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<tr>
<td></td>
<td>Age</td>
<td>Creep Coef.</td>
<td>$C(t)$</td>
<td>$\varepsilon_{sh}(t)$</td>
</tr>
<tr>
<td></td>
<td>days</td>
<td>$\phi(t)$</td>
<td>in$^2$/lb $\times 10^{-6}$</td>
<td>$\times 10^{-6}$</td>
</tr>
<tr>
<td>1 - max</td>
<td>18250</td>
<td>3.19</td>
<td>1.45</td>
<td>( )</td>
</tr>
<tr>
<td>1 - min</td>
<td>18250</td>
<td>2.31</td>
<td>0.97</td>
<td>( )</td>
</tr>
<tr>
<td>3 - max</td>
<td>18250</td>
<td>3.19</td>
<td>1.20</td>
<td>676</td>
</tr>
<tr>
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<td>2.27</td>
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<td>480</td>
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<td>18250</td>
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<td>( )</td>
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<tr>
<td>28 - min</td>
<td>18250</td>
<td>2.16</td>
<td>0.60</td>
<td>( )</td>
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</tbody>
</table>

Table 6.2: Recommended design value upper and lower bounds at 50 years. Based on 6"Ø×12" cylinders at 80% relative humidity.
Figure 6.1: Design creep coefficient recommendations for concrete loaded at 1 day.

Figure 6.2: Design specific creep recommendations for concrete loaded at 1 day.
Figure 6.3: Design creep coefficient recommendations for concrete loaded at 3 days.

Figure 6.4: Design specific creep recommendations for concrete loaded at 3 days.
Figure 6.5: Design creep coefficient recommendations for concrete loaded at 28 days.

Figure 6.6: Design specific creep recommendations for concrete loaded at 28 days.
Figure 6.7: Design shrinkage recommendations for concrete dried at 3 days and 80% relative humidity.
Bibliography


Appendix A

Material Property Data
Concrete Mix Design Supplied by the Contractor

<table>
<thead>
<tr>
<th>Material</th>
<th>Cement</th>
<th>Crushed Fine</th>
<th>3C (#67)</th>
<th>Water</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Source</td>
<td>Hawaiian</td>
<td>Glover</td>
<td>Glover</td>
<td>County</td>
<td></td>
</tr>
<tr>
<td>SDD Weight (lbs.)</td>
<td>752</td>
<td>1527</td>
<td>1505</td>
<td>324</td>
<td>4108</td>
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<tr>
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<td>2.61</td>
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<td>Absolute Volume (ft³)</td>
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<td>8.59</td>
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<td>26.19</td>
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<tr>
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<td>2.6</td>
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<td></td>
<td></td>
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<tr>
<td>% Absorption</td>
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<td>2.6</td>
<td></td>
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<tr>
<td>Correction %</td>
<td>0</td>
<td>0</td>
<td></td>
<td></td>
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<td>Correction (lbs.)</td>
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<td>0</td>
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<tr>
<td>Batch Weights (lbs.)</td>
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<td>1505</td>
<td>324</td>
<td>4108</td>
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<td>Daratard-17</td>
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<td>WRDA-HA</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>2 fl. oz./100 lbs cement</td>
<td>0.0 fl. oz./100 lbs cement</td>
<td>5 fl. oz./100 lbs cement</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Slump (in.) | 3 to 5 |
Air (%) | 3 |
Unit Weight (lbs.) | 152.2 |
W/C Ratio (lb/lb) | 0.43 |
Compressive Strength (psi) | 6000 |

Concrete Mix Design P1 & P2 - UH Concrete Technology Laboratory
12/19/98

<table>
<thead>
<tr>
<th>Material</th>
<th>Cement</th>
<th>Crushed Fine</th>
<th>3C (#67)</th>
<th>Water</th>
<th>Total</th>
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<td>Glover</td>
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<td></td>
<td>0.0 fl. oz./100 lbs cement</td>
<td>8.36 (fl-oz)</td>
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</tbody>
</table>

Slump (in.) | 1.5 |
Air (%) | 2 |
Unit Weight (lbs.) | 155.9 |
W/C Ratio (lb/lb) | 0.40 |

Figure A.1: Concrete mixture design for batches P1 & P2.
Concrete Mix Design Supplied by the Contractor

<table>
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<th>Water</th>
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<td>Glover</td>
<td>Glover</td>
<td>County</td>
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</tr>
<tr>
<td>SDD Weight (lbs)</td>
<td>752</td>
<td>1527</td>
<td>1505</td>
<td>324</td>
<td>4108</td>
</tr>
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<td>8.59</td>
<td>8.59</td>
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<td>26.19</td>
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<tr>
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</tr>
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<td>% Absorption</td>
<td>1.5</td>
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<td></td>
</tr>
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<td>Correction (lbs.)</td>
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<td>1505</td>
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<td>4108</td>
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Slump (in.) 3 to 5
Air (%) 3
Unit Weight (lbs.) 152.2
W/C Ratio (lb/lb) 0.43
Compressive Strength (psi) 6000

Concrete Mix Design P3 - UH Concrete Technology Laboratory
12/21/98

<table>
<thead>
<tr>
<th>Material</th>
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<td>Hawaiian</td>
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<td>0.0</td>
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<td>(fl-oz)</td>
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Slump (in.) 5.5
Air (%) 2
Unit Weight (lbs.) 152.2
W/C Ratio (lb/lb) 0.43

Figure A.2: Concrete mixture design for batch P3.
### KEALAKAHA BRIDGE DESIGN MIX P1 & P2 - DAY 1 TEST RESULTS

#### Concrete Properties

**Date Tested:** 12/20/97

<table>
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<tbody>
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<td>4022</td>
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<td></td>
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#### Modulus and Poisson's Ratio

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<td>% l', Load Stress Vert Gage Strain E_a Ave. E_a Horz Gage Strain v Ave. v</td>
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#### Summary

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<th>E_a (Ksi)</th>
<th>v</th>
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<td></td>
<td>4182</td>
<td>2834</td>
<td>0.208</td>
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</table>

Figure A.3: Test data for batches P1 & P2 at 1-day concrete age.
APPENDIX A. MATERIAL PROPERTY DATA

KEALAKAHA BRIDGE DESIGN MIX P1 & P2 - DAY 3 TEST RESULTS

Concrete Properties
Date Tested: 12/22/97

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<th>Cyl. No.</th>
<th>Pour</th>
<th>Top Dia. (in)</th>
<th>Bot. Dia. (in)</th>
<th>Area (sq. in.)</th>
<th>Length (in)</th>
<th>Weight (lb)</th>
<th>Density (pcf)</th>
<th>Load Cell (lb)</th>
<th>Rehile (lb)</th>
<th>f'c (psi)</th>
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</thead>
<tbody>
<tr>
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<td>6.0350</td>
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<td>6.0000</td>
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<td>31.02</td>
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<td>181750</td>
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<td>6.0000</td>
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<td>30.85</td>
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<td>30.82</td>
<td>156.13</td>
<td>6363</td>
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</table>

Modulus and Poisson's Ratio:

<table>
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<th>Cyl. No.</th>
<th>Pour</th>
<th>Top Dia. (in)</th>
<th>Bot. Dia. (in)</th>
<th>Area (sq. in.)</th>
<th>Length (in)</th>
<th>Weight (lb)</th>
<th>Density (pcf)</th>
<th>Cylinder 1 Modulus</th>
<th>Cylinder 1 Poisson Ratio</th>
<th>Cylinder 2 Modulus</th>
<th>Cylinder 2 Poisson Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>% I (%) Load (lb) Stress (psi) (x10^3 in)</td>
<td>Strain E (Ksi) Ave. E (Ksi) Horz Gage Strain (Ksi)</td>
<td>Ave. v</td>
<td>% I (%) Load (lb) Stress (psi) (x10^3 in)</td>
</tr>
<tr>
<td>1</td>
<td>P1</td>
<td>6.0469</td>
<td>6.0000</td>
<td>28.50</td>
<td>12.031</td>
<td>30.84</td>
<td>155.44</td>
<td>10 18000 632 26</td>
<td>-2</td>
<td></td>
<td>10 18000 632 26</td>
</tr>
<tr>
<td>2</td>
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<td>6.0469</td>
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<td>28.50</td>
<td>12.000</td>
<td>30.77</td>
<td>155.49</td>
<td>10 18000 632 27</td>
<td>-4</td>
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<td>10 18000 632 27</td>
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Summary:

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<th>f'c (psi)</th>
<th>E (Ksi)</th>
<th>v</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>6383</td>
<td>2873</td>
<td>0.223</td>
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Figure A.4: Test data for batches P1 & P2 at 3-day concrete age.
**KEALAKAHA BRIDGE DESIGN MIX P1 & P2 - DAY 28 TEST RESULTS**

**Concrete Properties:**

Date Tested: 1/19/96  (Actually 31 days)

<table>
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<th>Pour</th>
<th>Top Dia. (in)</th>
<th>Bot. Dia. (in)</th>
<th>Area (sq. in.)</th>
<th>Length (in)</th>
<th>Weight (lb)</th>
<th>Density (pcf)</th>
<th>Load Cell (lb)</th>
<th>Rehile (lb)</th>
<th>f_c' (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>P1</td>
<td>6.0468</td>
<td>6.0000</td>
<td>28.50</td>
<td>12.063</td>
<td>30.99</td>
<td>155.49</td>
<td>228150</td>
<td>225500</td>
<td>7960</td>
</tr>
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<td>P1</td>
<td>6.0468</td>
<td>6.0000</td>
<td>28.50</td>
<td>12.063</td>
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<td>155.59</td>
<td>225709</td>
<td>224500</td>
<td>7900</td>
</tr>
<tr>
<td>3</td>
<td>P2</td>
<td>6.0468</td>
<td>6.0000</td>
<td>28.50</td>
<td>12.063</td>
<td>30.95</td>
<td>155.29</td>
<td>226116</td>
<td>224500</td>
<td>7907</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>30.94</td>
<td>155.86</td>
<td>228150</td>
<td>225500</td>
<td>7922</td>
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**Modulus and Poisson's Ratio:**

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<th>Top Dia. (in)</th>
<th>Bot. Dia. (in)</th>
<th>Area (sq. in.)</th>
<th>Length (in)</th>
<th>Weight (lb)</th>
<th>Density (pcf)</th>
<th>Cylinder 1 Modulus</th>
<th>Cylinder 1 Poisson Ratio</th>
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</thead>
<tbody>
<tr>
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<td></td>
<td></td>
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<tr>
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<td>P1</td>
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<td>6.0000</td>
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<td>12.063</td>
<td>30.99</td>
<td>155.60</td>
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<td></td>
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<td>155.29</td>
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<table>
<thead>
<tr>
<th>% f_c'</th>
<th>Load Stress (lb)</th>
<th>Vert Gage (x10^4 in)</th>
<th>Strain (Ksi)</th>
<th>Ave. E_c (Ksi)</th>
<th>Horz Gage (x10^4 in)</th>
<th>Strain (Ksi)</th>
<th>Ave. v</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>22500 790 30</td>
<td>39</td>
<td>-5</td>
<td>-</td>
<td>-3</td>
<td>-</td>
<td>0.180</td>
</tr>
<tr>
<td>40</td>
<td>90000 3158 142</td>
<td>855</td>
<td>-19</td>
<td>-</td>
<td>-16.5</td>
<td>-0.000117</td>
<td>0.173</td>
</tr>
<tr>
<td>10</td>
<td>22500 790 30</td>
<td>39</td>
<td>-5</td>
<td>-</td>
<td>-3</td>
<td>-</td>
<td>0.000125</td>
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<tr>
<td>40</td>
<td>90000 3158 142</td>
<td>855</td>
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<td>-</td>
<td>-16.5</td>
<td>-0.000117</td>
<td>0.173</td>
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<table>
<thead>
<tr>
<th>% f_c'</th>
<th>Load Stress (lb)</th>
<th>Vert Gage (x10^4 in)</th>
<th>Strain (Ksi)</th>
<th>Ave. E_c (Ksi)</th>
<th>Horz Gage (x10^4 in)</th>
<th>Strain (Ksi)</th>
<th>Ave. v</th>
</tr>
</thead>
<tbody>
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<td>39</td>
<td>-5</td>
<td>-</td>
<td>-3</td>
<td>-</td>
<td>0.180</td>
</tr>
<tr>
<td>40</td>
<td>90000 3158 138</td>
<td>855</td>
<td>-12</td>
<td>-</td>
<td>-12</td>
<td>-0.000083</td>
<td>0.120</td>
</tr>
<tr>
<td>10</td>
<td>22500 790 27</td>
<td>39</td>
<td>-5</td>
<td>-</td>
<td>-3</td>
<td>-</td>
<td>0.000000</td>
</tr>
<tr>
<td>40</td>
<td>90000 3158 138</td>
<td>855</td>
<td>-12</td>
<td>-</td>
<td>-12</td>
<td>-0.000083</td>
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**Summary:**

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<th>Day 28</th>
<th>f_c' (psi)</th>
<th>E_c (Ksi)</th>
<th>v</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>7922</td>
<td>3510</td>
<td>0.180</td>
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* Inconsistent instrument readings. Value ignored.

Figure A.5: Test data for batches P1 & P2 at 28-day concrete age.
### KEALAKAHA BRIDGE DESIGN MIX P1 & P2 - DAY 56 TEST RESULTS

**Concrete Properties**

**Date Tested:** 2/13/98

<table>
<thead>
<tr>
<th>Cyl. No.</th>
<th>Pour</th>
<th>Top Dia.</th>
<th>Bot. Dia.</th>
<th>Area</th>
<th>Length</th>
<th>Weight</th>
<th>Density</th>
<th>Load Cell Reilte</th>
<th>f&lt;sub&gt;c&lt;/sub&gt;</th>
</tr>
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<tbody>
<tr>
<td></td>
<td></td>
<td>(in)</td>
<td>(in)</td>
<td>(sq. in.)</td>
<td>(in)</td>
<td>(lb)</td>
<td>(pcf)</td>
<td>(lb)</td>
<td>(psi)</td>
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<tr>
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<td>P1</td>
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<td>6.0000</td>
<td>28.42</td>
<td>12.083</td>
<td>30.87</td>
<td>155.59</td>
<td>241000</td>
<td>8479</td>
</tr>
<tr>
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<td>6.0000</td>
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<td>275000</td>
<td>9701</td>
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<td>5</td>
<td>P2</td>
<td>6.0313</td>
<td>6.0000</td>
<td>28.42</td>
<td>12.000</td>
<td>30.82</td>
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<td>277000</td>
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<td>7</td>
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<td>6.0000</td>
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**Modulus and Poisson's Ratio**

<table>
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<th>Pour</th>
<th>Top Dia.</th>
<th>Bot. Dia.</th>
<th>Area</th>
<th>Length</th>
<th>Weight</th>
<th>Density</th>
<th>Cylinder 1 Poisson Ratio</th>
<th>Cylinder 2 Poisson Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(in)</td>
<td>(in)</td>
<td>(sq. in.)</td>
<td>(in)</td>
<td>(lb)</td>
<td>(pcf)</td>
<td>% ε&lt;sub&gt;ε&lt;/sub&gt;</td>
<td>Load Stress</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>% ε&lt;sub&gt;ε&lt;/sub&gt;</td>
<td>(lb)</td>
</tr>
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<td>P1</td>
<td>6.0156</td>
<td>6.0000</td>
<td>28.35</td>
<td>12.047</td>
<td>30.86</td>
<td>156.18</td>
<td>0.232</td>
<td>3594</td>
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<tr>
<td>4</td>
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<td>6.0156</td>
<td>6.0000</td>
<td>28.35</td>
<td>12.016</td>
<td>30.88</td>
<td>156.66</td>
<td>0.232</td>
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### Summary:

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<th>E&lt;sub&gt;ε&lt;/sub&gt;</th>
<th>v</th>
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<tbody>
<tr>
<td></td>
<td>(psi)</td>
<td>(Ksi)</td>
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</tr>
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<td>9154</td>
<td>3645</td>
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</table>

Figure A.6: Test data for batches P1 & P2 at 56-day concrete age.
### APPENDIX A. MATERIAL PROPERTY DATA

#### KEALAKAHA BRIDGE DESIGN MIX P3- DAY 1 TEST RESULTS

**Concrete Properties**

- **Date Tested:** 1/22/98

**Compressive Strength:**

<table>
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<tr>
<th>Cyl. No.</th>
<th>Pour</th>
<th>Top Dia. (in)</th>
<th>Bot Dia. (in)</th>
<th>Area (sq. in.)</th>
<th>Length (in)</th>
<th>Weight (lb)</th>
<th>Density (pcf)</th>
<th>Load Cell (lb)</th>
<th>Reihle (lb)</th>
<th>fc' (psi)</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>P3</td>
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<td>6.0000</td>
<td>28.50</td>
<td>12.000</td>
<td>30.08</td>
<td>152.01</td>
<td>105000</td>
<td>105000</td>
<td>3685</td>
</tr>
<tr>
<td>2</td>
<td>P3</td>
<td>6.0469</td>
<td>6.0000</td>
<td>28.50</td>
<td>12.000</td>
<td>30.14</td>
<td>152.31</td>
<td>100750</td>
<td>100750</td>
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</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>30.11</strong></td>
<td><strong>152.16</strong></td>
<td><strong>105000</strong></td>
<td><strong>105000</strong></td>
<td><strong>3610</strong></td>
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**Modulus and Poisson's Ratio:**

<table>
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<th>Cyl. No.</th>
<th>Pour</th>
<th>Top Dia. (in)</th>
<th>Bot Dia. (in)</th>
<th>Area (sq. in.)</th>
<th>Length (in)</th>
<th>Weight (lb)</th>
<th>Density (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
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<td>6.0313</td>
<td>6.0000</td>
<td>28.42</td>
<td>12.188</td>
<td>30.20</td>
<td>150.66</td>
</tr>
<tr>
<td>4</td>
<td>P3</td>
<td>6.0313</td>
<td>6.0000</td>
<td>28.42</td>
<td>12.000</td>
<td>30.00</td>
<td>152.00</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Cylinder 1 Modulus</th>
<th>Load Stress</th>
<th>Vert Gage (x10^4 in)</th>
<th>Strain (Ksi)</th>
<th>Ave. E_o (Ksi)</th>
<th>Horz Gage (x10^4 in)</th>
<th>Strain</th>
<th>Ave. v</th>
</tr>
</thead>
<tbody>
<tr>
<td>% k' (%)</td>
<td>(lb)</td>
<td>(psi)</td>
<td>(Ksi)</td>
<td>(Ksi)</td>
<td>(psi)</td>
<td>(Ksi)</td>
<td>(Ksi)</td>
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**Summary:**

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<th>E_o (Ksi)</th>
<th>v</th>
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*Figure A.7: Test data for batch P3 at 1-day concrete age.*
### APPENDIX A. MATERIAL PROPERTY DATA

#### KEALAKAHA BRIDGE DESIGN MIX P3 - DAY 3 TEST RESULTS

**Concrete Properties**

**Date Tested:** 1/24/98

<table>
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<th>Concrete Properties</th>
<th>Strength</th>
<th>Dimensions</th>
</tr>
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<td></td>
<td></td>
</tr>
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<tr>
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<td>P3</td>
<td>6.0313</td>
</tr>
<tr>
<td><strong>Average:</strong></td>
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</table>

**Modulus and Poisson’s Ratio:**

<table>
<thead>
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<th><strong>Cyl. No.</strong></th>
<th><strong>Pour</strong></th>
<th><strong>Top Dia.</strong></th>
<th><strong>Bot. Dia.</strong></th>
<th><strong>Area</strong></th>
<th><strong>Length</strong></th>
<th><strong>Weight</strong></th>
<th><strong>Density</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>P3</td>
<td>6.0469</td>
<td>6.0000</td>
<td>28.50</td>
<td>12.000</td>
<td>30.08</td>
<td>152.01</td>
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</table>

**Cylinder 1 Modulus**

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<th>Vert Gage Strain</th>
<th>Ave. Ec’</th>
<th>Horz Gage Strain</th>
<th>V</th>
<th>Ave. v</th>
</tr>
</thead>
<tbody>
<tr>
<td>(%)</td>
<td>(lb) (psi) (x10⁴ in)</td>
<td>(Ksf) (Ksf)</td>
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<td></td>
<td></td>
<td></td>
</tr>
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<td>14100 495 14</td>
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</tr>
<tr>
<td>10</td>
<td>14100 495 22</td>
<td>-4</td>
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<td></td>
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<td>0.215</td>
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**Cylinder 1 Poisson Ratio**

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<th>Vert Gage Strain</th>
<th>Ave. Ec’</th>
<th>Horz Gage Strain</th>
<th>V</th>
<th>Ave. v</th>
</tr>
</thead>
<tbody>
<tr>
<td>(%)</td>
<td>(lb) (psi) (x10⁴ in)</td>
<td>(Ksf) (Ksf)</td>
<td></td>
<td></td>
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<td></td>
</tr>
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**Cylinder 2 Modulus**

<table>
<thead>
<tr>
<th>% Ec’</th>
<th>Load Stress</th>
<th>Vert Gage Strain</th>
<th>Ave. Ec’</th>
<th>Horz Gage Strain</th>
<th>V</th>
<th>Ave. v</th>
</tr>
</thead>
<tbody>
<tr>
<td>(%)</td>
<td>(lb) (psi) (x10⁴ in)</td>
<td>(Ksf) (Ksf)</td>
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<td></td>
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<td></td>
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<td>14100 494 14</td>
<td>-3</td>
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<tr>
<td>40</td>
<td>56900 1978 120</td>
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<td>2581</td>
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**Cylinder 3 Poisson Ratio**

<table>
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<th>Load Stress</th>
<th>Vert Gage Strain</th>
<th>Ave. Ec’</th>
<th>Horz Gage Strain</th>
<th>V</th>
<th>Ave. v</th>
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<tbody>
<tr>
<td>(%)</td>
<td>(lb) (psi) (x10⁴ in)</td>
<td>(Ksf) (Ksf)</td>
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<td>14100 494 14</td>
<td>-3</td>
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<td>56900 1978 115</td>
<td>0.000569 2600</td>
<td>-14</td>
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<td>0.176</td>
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<td>14100 494 26</td>
<td>2581</td>
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<td>40</td>
<td>56900 1978 119</td>
<td>0.000581 2553</td>
<td>-14</td>
<td>-0.000100</td>
<td>0.172</td>
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</tbody>
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**Summary:**

<table>
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<th>Day 3</th>
<th>fc’</th>
<th>Ec’</th>
<th>V</th>
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<tr>
<td></td>
<td>(psi)</td>
<td>(Ksf)</td>
<td>(Ksf)</td>
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<tr>
<td>4926</td>
<td>2659</td>
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Figure A.8: Test data for batch P3 at 3-day concrete age.
### APPENDIX A. MATERIAL PROPERTY DATA

#### KEALAKEHA BRIDGE DESIGN MIX P3 - DAY 28 TEST RESULTS

<table>
<thead>
<tr>
<th>Concrete Properties</th>
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<tbody>
<tr>
<td>Date Tested: 2/18/98</td>
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</table>

#### Compressive Strength:

<table>
<thead>
<tr>
<th>Cyl. No.</th>
<th>Pour</th>
<th>Top Dia.</th>
<th>Bot. Dia.</th>
<th>Area (sq. in.)</th>
<th>Length (in.)</th>
<th>Weight (lb)</th>
<th>Density (pcf)</th>
<th>Load Cell (lb)</th>
<th>Relative f′c (psi)</th>
<th>f′c (psi)</th>
</tr>
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<tbody>
<tr>
<td>3</td>
<td>P3</td>
<td>6.0468</td>
<td>6.0000</td>
<td>28.50</td>
<td>12.063</td>
<td>30.05</td>
<td>151.07</td>
<td>208500</td>
<td>206500</td>
<td>7247</td>
</tr>
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<td>4</td>
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<td>6.0000</td>
<td>28.50</td>
<td>12.063</td>
<td>30.35</td>
<td>152.58</td>
<td>212000</td>
<td>212000</td>
<td>7440</td>
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<tr>
<td>5</td>
<td>P3</td>
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<td>6.0000</td>
<td>28.50</td>
<td>12.063</td>
<td>30.15</td>
<td>151.57</td>
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<td>211000</td>
<td>7405</td>
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<tr>
<td>Average</td>
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<td>6.0000</td>
<td>28.50</td>
<td>12.063</td>
<td>30.18</td>
<td>151.74</td>
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<td>211000</td>
<td>7364</td>
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</table>

#### Modulus and Poisson's Ratio:

<table>
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<th>Cyl. No.</th>
<th>Pour</th>
<th>Top Dia.</th>
<th>Bot. Dia.</th>
<th>Area (sq. in.)</th>
<th>Length (in.)</th>
<th>Weight (lb)</th>
<th>Density (pcf)</th>
<th>Cylinder 1 Modulus</th>
<th>Cylinder 1 Poisson Ratio</th>
<th>Cylinder 2 Modulus</th>
<th>Cylinder 2 Poisson Ratio</th>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>E′ (Ksi)</td>
<td>v</td>
<td>E′ (Ksi)</td>
<td>v</td>
</tr>
<tr>
<td>1</td>
<td>P1</td>
<td>6.0468</td>
<td>6.0000</td>
<td>28.50</td>
<td>12.063</td>
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<td>152.23</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>2</td>
<td>P2</td>
<td>6.0468</td>
<td>6.0000</td>
<td>28.50</td>
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#### Summary:

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<th>f′c (psi)</th>
<th>E′ (Ksi)</th>
<th>v</th>
</tr>
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<td>7364</td>
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Figure A.9: Test data for batch P3 at 28-day concrete age.
### Modulus of Rupture

#### Rupture Tests 28-Day:

<table>
<thead>
<tr>
<th>Beam No.</th>
<th>Load (lb)</th>
<th>P=Load/2 (lb)</th>
<th>a (in)</th>
<th>M (in-lb)</th>
<th>y (in)</th>
<th>l (in4)</th>
<th>fr (psi)</th>
<th>Factor</th>
<th>Extreme Tensile Fiber Stress</th>
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<tbody>
<tr>
<td>P1</td>
<td>10100</td>
<td>5050</td>
<td>6</td>
<td>30300</td>
<td>3</td>
<td>108</td>
<td>842</td>
<td>9.5</td>
<td>=9.5'(fc)(^{1/2})</td>
</tr>
<tr>
<td>P2</td>
<td>10100</td>
<td>5050</td>
<td>6</td>
<td>30300</td>
<td>3</td>
<td>108</td>
<td>842</td>
<td>9.5</td>
<td>=9.5'(fc)(^{1/2})</td>
</tr>
<tr>
<td>P1</td>
<td>9025</td>
<td>4513</td>
<td>6</td>
<td>27075</td>
<td>3</td>
<td>108</td>
<td>752</td>
<td>8.4</td>
<td>=8.4'(fc)(^{1/2})</td>
</tr>
<tr>
<td>P2</td>
<td>9050</td>
<td>4525</td>
<td>6</td>
<td>27150</td>
<td>3</td>
<td>108</td>
<td>754</td>
<td>8.5</td>
<td>=8.5'(fc)(^{1/2})</td>
</tr>
</tbody>
</table>

#### Rupture Tests 28-Day:

<table>
<thead>
<tr>
<th>Beam No.</th>
<th>Load (lb)</th>
<th>P=Load/2 (lb)</th>
<th>a (in)</th>
<th>M (in-lb)</th>
<th>y (in)</th>
<th>l (in4)</th>
<th>s (psi)</th>
<th>Factor</th>
<th>Extreme Tensile Fiber Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>P3</td>
<td>7500</td>
<td>3750</td>
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<td>22500</td>
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<td>108</td>
<td>625</td>
<td>7.28</td>
<td>=7.3'(fc)(^{1/2})</td>
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<td>546</td>
<td>6.36</td>
<td>=6.4'(fc)(^{1/2})</td>
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</table>

#### Rupture Tests 56-Day:

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<th>Beam No.</th>
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<th>P=Load/2 (lb)</th>
<th>a (in)</th>
<th>M (in-lb)</th>
<th>y (in)</th>
<th>l (in4)</th>
<th>s (psi)</th>
<th>Factor</th>
<th>Extreme Tensile Fiber Stress</th>
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<tbody>
<tr>
<td>P1</td>
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<td>4225</td>
<td>6</td>
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<tr>
<td>P2</td>
<td>8750</td>
<td>4375</td>
<td>6</td>
<td>26250</td>
<td>3</td>
<td>108</td>
<td>729</td>
<td>7.62</td>
<td>=7.6'(fc)(^{1/2})</td>
</tr>
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Figure A.10: Concrete tensile strength results from modulus of rupture beam tests.
Appendix B

Creep Data
Figure B.1: Measured creep coefficient, $\phi(t)$, for batches P1, P2 & P3 loaded at all concrete ages.
Creep per Axial Stress
P1, P2 & P3

Figure B.2: Measured creep per axial stress, for batches P1, P2 & P3 loaded at all concrete ages.
Figure B.3: Creep coefficient, \( \phi(t) \), for batches P1 & P2 loaded at 1-day.
Figure B.4: Regression analysis for batches P1 & P2 loaded at 1-day.
Figure B.5: Creep coefficient, $\phi(t)$, for batches P1 & P2 loaded at 1-day.
Figure B.6: Five year creep coefficient, $\phi(t)$, for batches P1 & P2 loaded at 1-day.
1 Day - Creep Coefficient
P1 & P2 (50 Years)

Concrete Age (Days)

φ Coefficient

3.50 3.00 2.50 2.00 1.50 1.00 0.50 0.00

- 1 Day
- Bazant (1 Day)
- Mod. Bazant (1 Day/50)

Figure B.7: Fifty year creep coefficient, \( \phi(t) \), for batches P1 & P2 loaded at 1-day.
Figure B.8: Creep coefficient, $\phi(t)$, for batches P1 & P2 loaded at 3-days.
Figure B.9: Regression analysis for batches P1 & P2 loaded at 3-days.
Figure B.10: Five year creep coefficient, $\phi(t)$, for batches P1 & P2 loaded at 3-days.
3 Day - Creep Coefficient
P1 & P2 (50 Years)

Figure B.11: Fifty year creep coefficient, $\phi(t)$, for batches P1 & P2 loaded at 3-days.
Figure B.12: Creep coefficient, $\phi(t)$, for batches P1 & P2 loaded at 28-days.
Figure B.13: Regression analysis for batches P1 & P2 loaded at 28-days.
Figure B.14: Five year creep coefficient, $\phi(t)$, for batches P1 & P2 loaded at 28-days.
Figure B.15: Fifty year creep coefficient, $\phi(t)$, for batches P1 & P2 loaded at 28-days.
Figure B.16: Creep coefficient, $\phi(t)$, for batch P3 loaded at 3-days.
Figure B.17: Regression analysis for batch P3 loaded at 3-days.
Figure B.18: Five year creep coefficient, $\phi(t)$, for batch P3 loaded at 3-days.
3 Day - Creep Coefficient
P3 (50 Years)

Figure B.19: Fifty year creep coefficient, $\phi(t)$, for batch P3 loaded at 3-days.
Creep Coefficient
P1, P2 & P3 (5 Years)

Figure B.20: Five year creep coefficient, $\phi(t)$, for batches P1, P2, & P3.
Figure B.21: Five year creep per axial strain, $C(t)$, for batches P1, P2, & P3.
Figure B.22: Five year creep coefficient, $\phi(t)$, for batches P1, P2, & P3 including estimates.
Figure B.23: Five year creep per axial strain, $C(t)$, for batches P1, P2, & P3 including estimates.
Figure B.24: Fifty year creep coefficient, $\phi(t)$, for batches P1, P2, & P3 including estimates.
Figure B.25: Fifty year creep per axial strain, C(t), for batches P1, P2, & P3 including estimates.
Appendix C

Shrinkage Data
Shrinkage Data
P1, P2 & P3

Figure C.1: Measured shrinkage for batches P1, P2 & P3 at all concrete ages.
Figure C.2: Shrinkage for batches P1 & P2 dried at 1-day.
Figure C.3: Regression analysis for batches P1 & P2 dried at 1-day.
Figure C.4: Shrinkage for batches P1 & P2 dried at 1-day.
Figure C.5: Five year shrinkage for batches P1 & P2 dried at 1-day.
Figure C.6: Fifty year shrinkage for batches P1 & P2 dried at 1-day.
3 Day - Shrinkage
P1 & P2

Figure C.7: Shrinkage for batches P1 & P2 dried at 3-days.
Figure C.8: Regression analysis for batches P1 & P2 dried at 3-days.
Figure C.9: Five year shrinkage for batches P1 & P2 dried at 3-days.
Figure C.10: Fifty year shrinkage for batches P1 & P2 dried at 3-days.
Figure C.11: Shrinkage for batches P1 & P2 dried at 28-days.
Figure C.12: Regression analysis for batches P1 & P2 dried at 28-days.
Figure C.13: Five year shrinkage for batches P1 & P2 dried at 28-days.
Figure C.14: Fifty year shrinkage for batches P1 & P2 dried at 28-days.
Figure C.15: Shrinkage for batch P3 dried at 3-days.
Figure C.16: Regression analysis for batch P3 dried at 3-days.
Figure C.17: Five year shrinkage for batch P3 dried at 3-days.
Figure C.18: Fifty year shrinkage for batch P3 dried at 3-days.
Figure C.10: Five year shrinkage for batches P1, P2, & P3.
Shrinkage
P1, P2 & P3 (50 Years)

Figure C.20: Fifty year shrinkage for batches P1, P2, & P3.