PRELIMINARY ASSESSMENT OF

THE BASE VARIABLES FOR STANDARDIZING

THE PRESSURE TENSION TEST

Aifei Lu

260329941

Department of Civil Engineering and Applied Mechanics

McGill University

Montreal, Canada

February 2015

A thesis submitted to McGill University in partial fulfillment of the requirements of the degree of Master of Engineering

TABLE OF CONTENT

LIST OF FIGURES

LIST OF TABLES

ABSTRACT

The tensile strength of concrete plays a critical role in its cracking mechanics, and is therefore one of the most important properties of concrete. Existing tests for direct tension tend to induce secondary stresses, while common indirect tension tests such as the splitting tension test and the flexural test, provide less accurate estimates of tensile strength. The pressure tension test is a newly developed indirect test method that has been shown to be a good indicator of the true tensile strength of concrete. The assay applies an axisymmetrical compressive pressure onto the curved surface of a concrete cylinder, indirectly creating a pure uniaxial tensile stress field within the tested specimen that is free of induced secondary stress. For this reason, pressure tensile strength is expected to become a standard specification criterion, alongside compressive strength, in future concrete designs.

The objective of this research was to assess the base variables required to establish a standard testing method for the pressure tension test. The scope of the project also included preparing an initial draft for the standard testing method. Concrete specimens were cast at three different W/C and tested at varying moisture content, rates of stress, and creep conditions to evaluate the effects associated with these base parameters. The variability inherent in the test method itself was also investigated. A base stress rate of 3 psi/s [0.021 MPa/s] was used in this experiment, which is comparable to the standard loading rate in the splitting tension test (ASTM C 496). The low coefficient of variation in the pressure tensile strength results (10-16%) is comparable to the splitting tension test and demonstrates the high reliability of the method. Results also show that the pressure tensile strength is very sensitive to the moisture content in concrete, which confirms previous pressure tension studies. Large decreases in strength were observed when specimens were allowed to dry, even for short periods of time. Failure stress measurements at stressing rates between 1 psi/s [0.007 MPa/s] and 5 psi/s [0.034 MPa/s] did not vary significantly, hence it can be deduced that the test method is repeatable within this stress rate range and that minor fluctuations can be tolerated. This research has also demonstrated the capability of the pressure tension test to maintain a constant applied stress

for an extended period of time, allowing for the possibility of performing tension creep testing for concrete.

SOMMAIRE

La résistance à la traction du béton joue un rôle essentiel dans sa mécanique de rupture, et elle est donc l'une des propriétés les plus importantes de béton. Les essais de traction directs existants ont tendance d'induire un contrainte secondaire, alors que les essais de traction indirects communs tels que l'essai de la résistance en traction par fendage et l'essai de la résistance à la flexion de poutre béton fournissent des estimations moins précises de la résistance à la traction. L'essai de la résistance à la traction de pression est une méthode indirecte nouvellement développée qui est démontré à être un bon indicateur de la véritable résistance à la traction du béton. L'essai s'applique une charge de compression axisymétrique sur un cylindre de béton, créant indirectement un champ de contrainte de traction uniaxiale pure et libre de contrainte secondaire dans l'échantillon testé. Pour cette raison, la résistance à la traction de pression est prévu pour devenir un critère de spécification standard, aux côtés de la résistance à la compression, dans les conceptions futures du mélange de béton.

L'objectif de cette recherche était d'évaluer les variables de base nécessaires pour établir la méthode standard d'essai pour l'essai de traction de pression. La portée du projet comprend également la préparation d'un brouillon préliminaire pour la méthode standard d'essai. Des échantillons de béton ont été coulés en trois différents rapports eau-ciment et ils ont été testés à différentes teneurs en eau, taux de contrainte, et conditions de fluage pour évaluer les effets associés à ces paramètres de base. La variabilité dans la méthode d'essai a également été déterminée. Un taux de contrainte de base de 3 psi/s [0.021 MPa/s] est utilisé dans cette expérience, ce qui est comparable au taux de charge standard dans l'essai de traction de fendage (ASTM C496). Le faible coefficient de variation des résultats de contraintes (10-16%) est comparable à l'essai de traction de fendage et démontre la fiabilité de la méthode. Les résultats montrent également que la résistance à la traction de pression de béton est très sensible à la teneur en eaux, ce qui confirme les études précédents. Fortes baisses de la résistance à la traction ont été observés lorsque les échantillons sont laissés à sécher, même pour une courte période de temps. Mesures de contrainte à la rupture à des taux de contrainte

entre 1 psi/s [0.007 MPa/s] et 5 psi/s [0.034 MPa/s] ne varient pas significativement, donc on peut en déduire que la méthode d'essai est reproductible dans cette gamme de taux de contrainte et que les fluctuations mineures peuvent être tolérées. Cette recherche a également démontré la capacité de l' essai de traction de pression de maintenir un contraint appliqué constant pour une durée prolongée, permettant la possibilité d'effectuer des essais de fluage de traction pour le béton.

ACKNOWLEDGMENT

My appreciations to Dr. Andrew Boyd for the opportunity to carry out this project and for his ongoing guidance in completing this thesis. Special thanks to Andrew Komar for continually providing crucial feedback and sharing his methodological expertise, as well as developing and continually improving the software required for the pressure tension apparatus, and to Charles-Eric Desjardins for his exceptional experimental assistance. I would also like to acknowledge the rest of my research team, Julie-Anne Hartell, Dominique Langelier, and Omar Shaikhon, for their support and collaboration on this project.

Many thanks to John Bartczak and Gerard Bechard for their technical assistance and work towards ensuring instrument function and safety at all time during the period of study.

I would also like to recognize Lafarge Canada Inc. and Bauval Inc. for providing the necessary materials for casting the concrete specimens, without which this research would not be possible.

Lastly, I would like to convey my greatest thanks and gratitude to my family and friends for their love and support during the course of my Master's thesis.

1. INTRODUCTION

Concrete is the most widely used construction material in the world. The application and use of the composite is highly dependent on its properties as defined by its mixture proportion and the method of preparation. Compressive strength is the most common measure of the overall quality of the concrete, and is generally accepted as the most direct indication of the material's ability to resist loads in structural applications (Chen et al., 2012; Bungey et al., 2006; Haroun, 1968). Compressive strength testing is relatively easy to conduct, and the results are often correlated to other properties of concrete that often require more complicated methods of testing (Ozyildirim & Carino, 2006).

The use of strength tests for concrete has three main purposes, which include research, quality control and quality assurance, and in-place concrete strength assessment. In research, these tests measure the effect of different mixture ingredients and proportions on concrete strength. The measurements are also used as reference values for the study of other concrete properties. In terms of quality control in construction, strength tests evaluate the adequacy of the mixture proportions developed for specific jobs. Quantified changes in strength could be an indicator of changes in ambient conditions. On-site strength tests also provide information on whether the concrete in the structure is cured sufficiently for the application of construction loads, for the removal of framework, or for the application of pre-stressing.

Unlike certain material properties, concrete strength is not an absolute property. Results vary depending on the shape and size, specimen preparation, and the loading method. Therefore, strength tests require extensive standardization for reliable use in the field.

The pressure tension test is a new method for determining concrete tensile strength. Originally developed at the Building Research Establishment in the United Kingdom with water as loading fluid, the test involves subjecting a concrete cylinder to indirect tension through the application of axisymmetrical stress.

The objective of this study is to establish a standardized testing method for the pressure tension test. The scope of the project included preparing an initial draft standard procedure, as well as assessing the required base variables and defining an initial set of test conditions as a control for future development. In contrast to previously documented pressure tension tests which employed loading fluids such as water and nitrogen gas (Clayton & Grimmer, 1979), the technique in this research utilized compressed air. Gas loading fluids are more permeable and provide more reliable results as compared to liquid loading fluids (Clayton, 1978). Parameters required for the standardization of the pressure tension test were evaluated. The experimental program was divided into four series of testing:

(1) Evaluation of the variability inherent in the test method

(2) Evaluation of the effects of moisture content in the concrete

(3) Evaluation of the effects of stress rate

(4) Evaluation of the effects of creep

Each series consisted of tests on three different mixture proportions for means of comparison. The purpose of these standardization assays was to maximize compatibility, interoperability, safety, repeatability, and quality of this novel concrete tensile testing technique. The pressure tensile strength is expected to become a specification criterion in future concrete designs, alongside the compressive strength.

This thesis includes a literature review in Chapter 2 that discusses the various recognized concrete tensile strength tests, the effects of various parameters on the test results, and the theory behind the pressure tension test. Chapter 3 will describe the details and procedures of the experimental program. The results of the tests are then compared and analyzed in Chapter 4. Finally, Chapter 5 consists of the proposed standard test method for the pressure tensile strength of cylindrical concrete specimens.

2. LITERATURE REVIEW

This chapter reviews the different methods for determining the tensile strength of concrete. The advantages and disadvantages of each method are detailed, as well as the effects of altering various test parameters. This chapter also explains the mechanics behind the pressure tension test and amalgamates previous research pertaining to the test method and mechanism.

2.1. Concrete Tensile Strength

When considering the strength of concrete, compressive strength is typically considered as the most relevant construction property and a measure of the overall quality of the concrete (Li, 2004). Tensile strength is less commonly considered, but it has a critical role to play in the fracture mechanism of concrete. The measure of compressive strength is unable to account for this because the fracture of concrete is initiated through cracking. Cracking is a form of tension failure, whether failure is due to an applied load, such as compression, or other factors. Cracking of concrete is the most common type of failure in deteriorating concrete structures (Bremner et al., 1995) and it has major importance in some structures such as concrete pavements or dams. There has been increasing recognition of tensile strength as an important property to be considered (Ozyildirim & Carino, 2006).

Concrete is a brittle composite consisting of hydrated cement paste and aggregate. The interfacial transition zones formed between the constituents are generally weaker than either of the constituents alone. Microcracks are generated during the hydration process at the interfacial transition zones due to mechanical property differences between the composite constituents, as well as shrinkage and thermal strains. These pre-existing microcracks are the primary reason for the low tensile strength of concrete. They are generally stable up to an external load of 30% of the ultimate load (Neville, 1996). Beyond this magnitude, these cracks increase in length and width, as well as in quantity to a certain degree, contributing to continual loss of material integrity and decrease in tensile strength. The leading edge of cracks is generally in the form of multiple branching microcracks. These microcracks coalesce gradually into a single macrocrack as the crack progresses. (Li, 2004) At 70% to 90% of ultimate load, the cracks start to penetrate the bulk paste to form larger cracks (Neville, 1996).

In contrast to compressive loading conditions, where pre-existing cracks and defects are closed up (Chen et al., 2012), tensile loads induces a greater propensity for cracks to propagate. In fact, the tensile strength of concrete is estimated to be about 10 to 15% of the compressive strength (Lin & Wood, 2003; Li, 2004). Due to the weak tensile strength and the brittle nature of concrete, the composite is rarely designed to resist direct tension. Nonetheless, tensile strength measures are necessary to determine resistance to cracking, as well as to specify proper steel reinforcement.

2.2. Concrete Tension Tests

Multiple tests determining the tensile strength of concrete exist and are used in practice. Each has its specific characteristics and differences, and they do not always correlate well with one another (Bremner et al., 1995). It is therefore important to consider the use and load application of the concrete when selecting the tension test to be performed for increased significance and reliability of results.

2.2.1. Direct Tensile Strength Tests

Direct tensile strength tests involve applying a pure tensile force along a single axis on the testing specimen. Even with enormous care during preparation, the direct tensile strength is expected to be 10 to 30% lower than the true tensile strength of concrete (Bremner et al., 1995). Although direct tension tests in themselves are time efficient and generally simple to conduct, sample preparation is extensive and time-consuming and their repeatability is often questioned by most researchers (Bolzan & Huber, 1993). Consistent test replications are difficult to achieve due to the introduction of localized stress concentrations when pure tensile force is applied. A uniform state of stress is extremely difficult to obtain (Haroun, 1968; van Mier & van Vliet, 2002).

Multiple variations of the direct tension test have been attempted in order to eliminate stress concentrations (Bremner et al., 1995). The variations can be categorized into three major groups: tests that grip the ends of the specimens, tests that connect the specimen end surfaces to the test machine with adhesives, and tests that apply load via an embedded steel bar. So far, no standards have been recognized for any direct tension test (ACI 224, 1997; Ozyildirim & Carino, 2006).

In contrast to direct tension tests, indirect tension tests have been studied extensively due to their simplicity, specifically the splitting tension test and the flexural test. Nonetheless, there is some evidence that, if performed properly, direct tension test results could be more reliable (Swaddiwudhipong et al., 2003) and are the most accurate measure of concrete tensile strength (Aerllano & Thmpson, 1998).

2.2.1.1. Gripped Ends

In 1928, one of the earliest variations of the direct tension test was performed by Gonnerman and Shuman (1928) in which a conventional concrete cylinder is gripped by cylindrical steel straps friction-clamped at its ends. The grips apply a compressive force onto the concrete surface and grab the concrete specimen by the force of friction. A tensile force is then applied via the grips. The problem with this method is the development of concentrated stresses in the region of the grips (Bremner et al., 1995; van Mier & van Vliet, 2002), which leads to inaccurate measurements.

A test method similar to the ASTM D 2936 standard test method for direct tensile strength of intact rock core specimens (Figure 2-1) is sometimes used. This test applies axial tensile force through metal caps bonded to the ends of the specimen, and requires enormous care during preparation to prevent eccentric loading (Ozyildirim & Carino, 2006; van Mier & van Vliet, 2002).

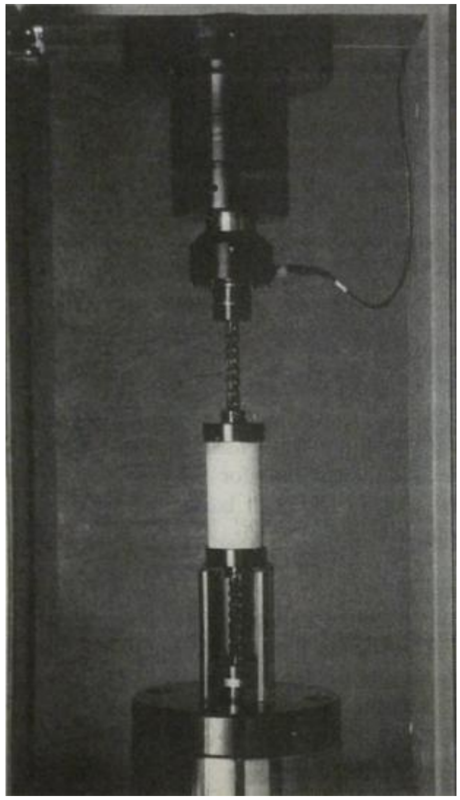


Figure 2-1 - Standard Test Method for Direct Tensile Strength of Intact Rock Core Specimens (ASTM D 2936)

Other attempts to optimize the gripped end direct tension method have required custom-fabricated specimens, such as bobbin-shaped specimens (Figure 2-2) or dogbone-shaped specimens (Figure 2-3). With bobbin-shaped specimens, the load is applied through a precision-made steel cap. A quick-setting cement paste is applied between the concrete and the steel cap to provide a good bond. The specimen is placed within a mould and a capping jig assembly to reduce eccentricity. The direct tensile force is then applied through a dead weight lever arm system (Neville et al., 1983). In the case of dogbone-shaped specimens, the grips clamp onto tapered aluminum plates which are glued on two lateral sides of each end of the specimen (Graybeal and Baby, 2013).

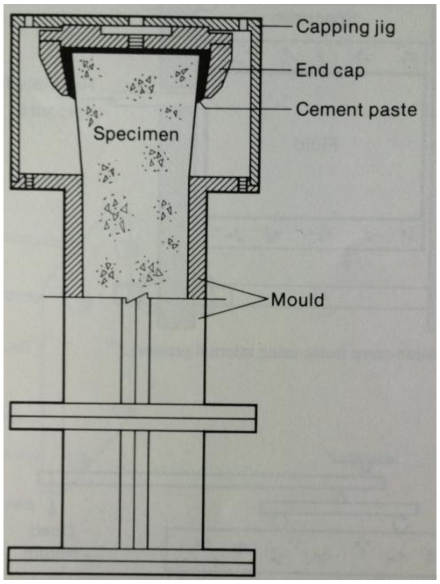


Figure 2-2 - Bobbin-Shaped Specimen (Nevile et al., 1983)

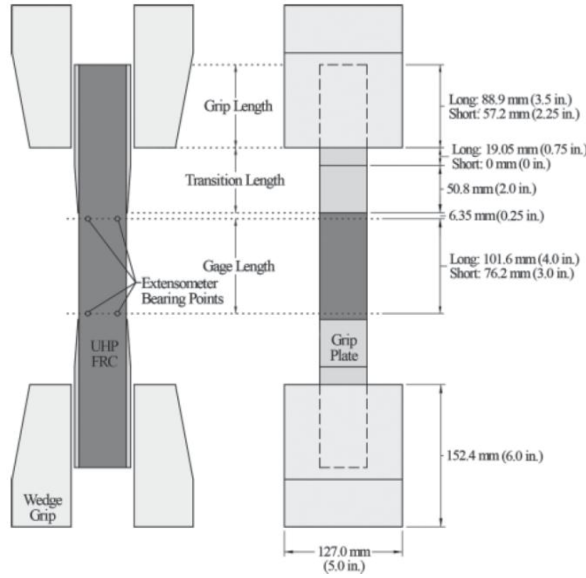


Figure 2-3 - Dogbone-Shaped Specimen (Graybeal and Baby, 2013)

2.2.1.2. Glued Ends

Gluing steel plates onto the ends of concrete specimens with high-strength epoxies (Figure 2-4) is another class of direct tension tests. Tensile force is then applied to loading rods that are bolted onto the steel plates. Loading alignment and non-uniformity of stress are the major problems with this method (Bremner et al., 1995). The main advantage of this method is that bending stresses will not have much influence on the results. On the other hand, premature non-uniform failures caused by local stress concentration near the ends of the specimen were observed (Graybeal & Baby, 2013; Bolzan & Huber, 1993). Moreover, non-adherence of the steel plates to moist concrete can be problematic (Swaddiwudhipong et al., 2003).

Figure 2-4 - Direct Tension Test with Glued Ends (Bolzan & Huber, 1993)

2.2.1.3. Embedded Steel Bar

The embedded steel bar variation of the direct tension test involves applying tensile force to the concrete specimen through an embedded steel reinforcing bar (Figure 2-5). When a tensile force is applied to the steel bar, the bond between the embedded steel and the concrete matrix transfers a partial load onto the concrete. Attached strain gauges record the strain, which is used to calculate the load carried by the steel. The tensile strength of the concrete is then determined by deducing the load carried by the steel bar from the failure load.

Two major problems exist for this method. The first issue is the lack of existing measures to ensure a proper bond development between the steel bar and the concrete matrix (Bremner et al., 1995; Swaddiwudhipong et al., 2003). A good bond between the two materials is required to ensure the same strain and thus the proper transfer of load from the steel to concrete. A second problem is the difficulty in centering and aligning the steel bar. Miscentering and misalignment induces eccentricities on the specimen. High stress concentrations have also been observed in this method to cause fracture at the ends of the specimens (Swaddiwudhipong et al., 2003).

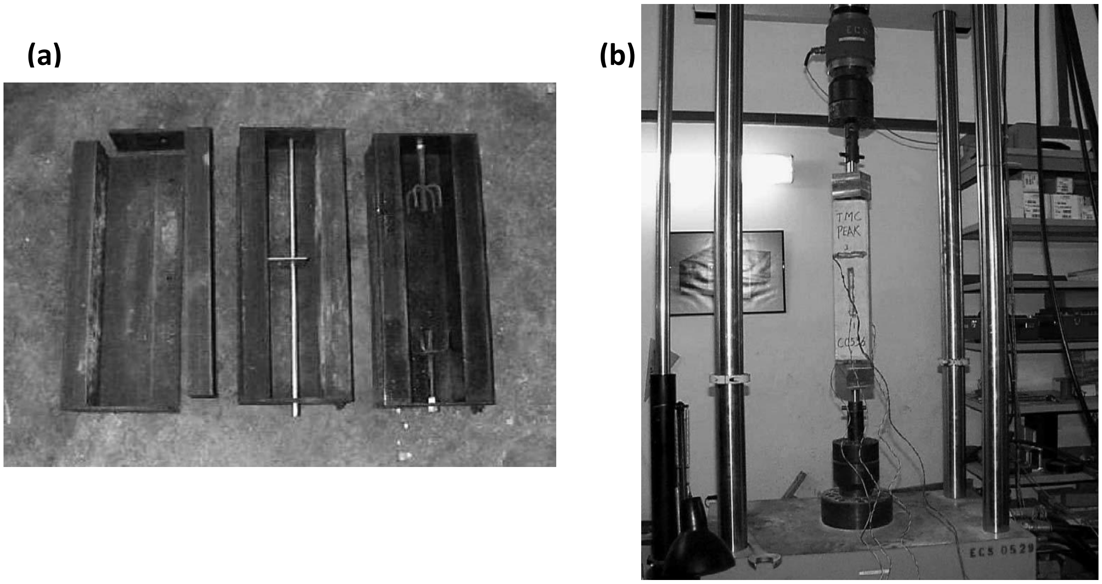


Figure 2-5 - Direct Tension Test using Embedded Steel Bar
(a) Mould for Embedding Steel (b) Test Setup
(Swaddiwudhipong et al., 2003)

2.2.2. Flexural Strength Tests

The flexural strength test simulates stress conditions found in horizontal bending members, such as beams or pavements. In this test, a beam specimen supported at its ends is subjected to one or more perpendicular loads until failure occurs. ASTM currently defines two flexural tests, which include the simple beam with center-point loading (ASTM C 293), and the simple beam with third-point loading (ASTM C 78).

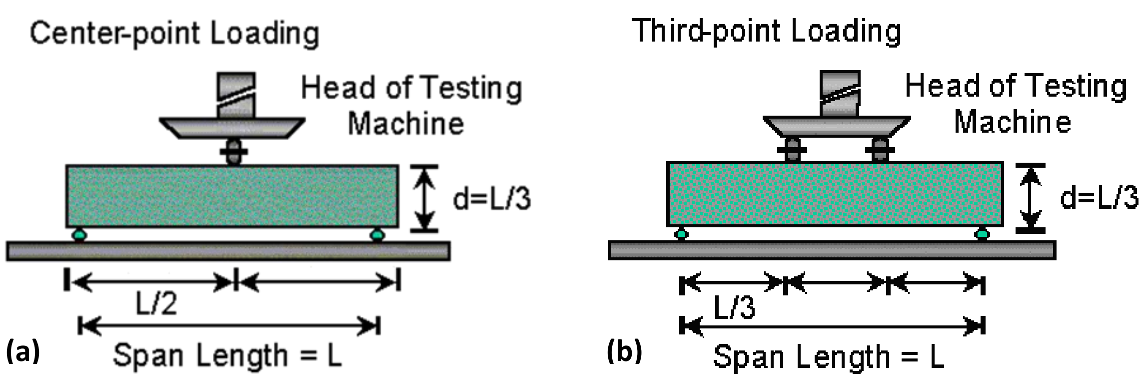


Figure 2-6 - Flexural Strength Tests
(a) Simple Beam with Center-Point Loading (ASTM C 293) (b) Simple Beam with Third-Point Loading (ASTM C 78)

The center-point loading flexural test consists of one point load at the center of the concrete beam (Figure 2-6 (a)), thus creating a maximum bending moment only at the loading location. On the other hand, the third-point loading flexural test subjects the specimen to two equal point loads placed at the third points of the specimen span (Figure 2-6 (b)). This induces a uniform maximum bending moment along the middle third of the beam. The loading conditions of the flexural test creates a non-uniform state of stress. In both test conditions, direct tension at the bottom face of the beam caused by the bending moment initiates cracking and failure of the specimen. The tensile stress at this point is calculated from basic beam theory, and assumed to be the tensile strength of the concrete.

Since maximum stress occurs only at the central point for center-point loading, defects at other regions that would reduce the tensile strength might not manifest or cause the ultimate failure. With third-point loading conditions, a greater span of the specimen would undergo the maximum moment, providing a higher probability to encounter the weakest point within the specimen. As a result, third-point loading method often gives flexural strengths that are 15% lower than with center-point loading (Carrasquillo & Carrasquillo, 1987). Since the third-point loading flexural test puts more concrete under effective test, the results are more consistent and are likely to be closer to the true tensile strength of concrete.

Flexural strength is usually measured to be 40-80% higher than splitting tensile strength (ACI 224, 1997), a consequence of the simple flexure formula's assumption that concrete behaves as a linear-elastic material throughout the test. However, concrete has a nonlinear stress-strain curve, and the elastic behavior does not persist until failure (Arellano & Thompson, 1998). Moreover, the variability of the flexural test tends to be relatively high, due to its sensitivity to conditions of fabrication, handling, curing, and testing (Ozyildirim & Carino, 2006).

As with compressive strength, the flexural strength of the material increases linearly with loading rate (Ozyildirim & Carino, 2006). Flexural strength is also very sensitive to moisture conditions (Johnston & Sidwell, 1969; Chen et al., 2002). The apparent flexural strength of drying concrete specimens can be lower than that of saturated specimens by as much as 33% (Nielsen, 1954). Surface shrinkage due to rapid drying of the surface induces tensile stress, which initiates surface cracks when this stress exceeds the concrete tensile strength (van Mier & van Vliet, 2002; Haroun, 1968).

2.2.3. Splitting Tensile Strength Tests

As a result of the unreliability of direct tension tests and the limitations of other previously known indirect test methods, the splitting tension test was proposed by Fernando Carneiro in 1943 (Fairbairn & Ulm, 2002). The splitting tension test, also known as the Brazilian test, is an indirect tensile test that can be applied to all materials. It is currently the most recognized concrete tensile strength test in the world (Fairbairn & Ulm, 2002), and it is included in several testing standards including both ASTM (ASTM C 496) and CSA (CSA A23.2-13C).

The splitting tension test involves applying a uniform diametrical compressive force along the length of a concrete cylinder (Figure 2-7 (a)). This loading creates a uniform tensile stress along most, but not the entirety, of the diameter (Figure 2-7 (b)), as per the theory of elasticity (Arιoglu et al., 2006). The specimen reaches failure by splitting along the diametrical loading plane (Figure 2-8).

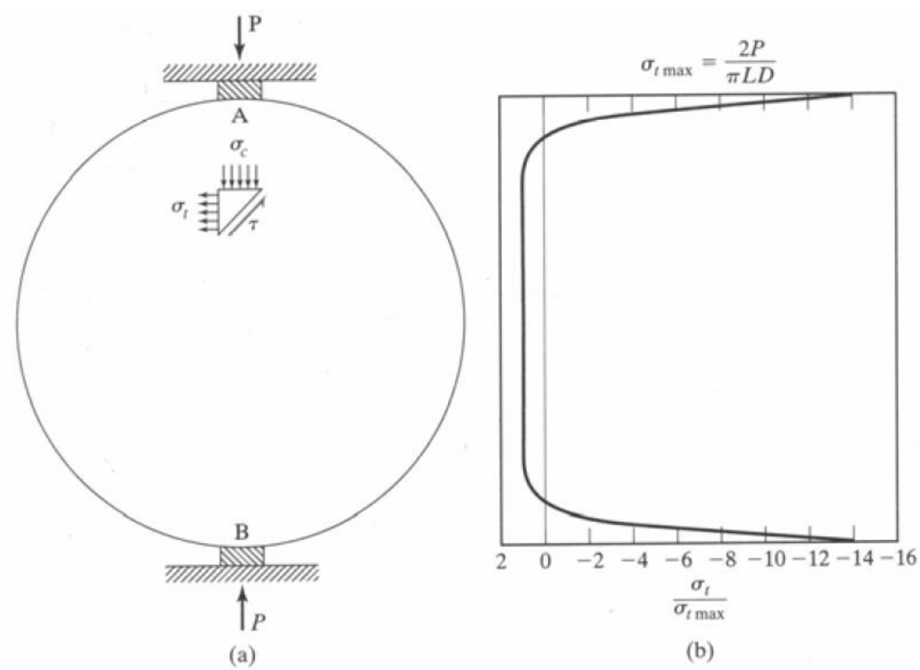


Figure 2-7 - Splitting Tension Test
(a) Loading Diagram (b) Tensile Stress Distribution along the Vertical Diameter of the Specimen (Mindess et al. 2002)

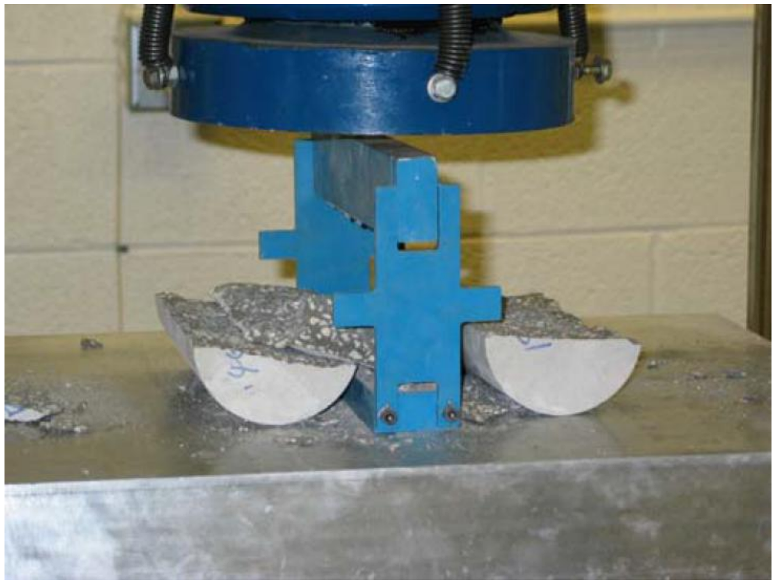


Figure 2-8 - Specimen failing under Splitting Tension Test
(Li, 2004)

However, although the failure mechanism of this test method is tension, the result is not direct evidence of the true tensile strength of concrete. As stated by ASTM C 496, splitting tensile strength of concrete is expected to be larger than the direct tensile strength, by approximately 10% (Hannant et al., 1973), but lower than the flexural strength, given that the flexural strength is about 40% to 80% higher (ACI 224, 1997). Its applicability for estimating the uniaxial tensile strength has also been questioned by researchers, due to the differences between the theoretical model and the true set-up of the test. The applied compressive load creates a stress field within the material that is not purely uniaxial tension (Malarics & Muller, 2010), but also includes compressive stress and other stress states (van Mier & van Vliet, 2002). Moreover, the splitting tension test does not take into account the critical defects within the composite, since the load is only applied along a fixed narrow strip of the concrete specimen. There is a high probability that flaws are not located along the central plane where the load is applied. Another disadvantage of this test method is that its loading conditions do not resemble those in the field (Hudson & Kennedy, 1968).

On the other hand, the Brazilian test is very simple to perform, as it only requires a standard compression testing machine and a concrete cylinder identical to those used for the compression test. Also, a major advantage of the method compared to the flexural test is that the tensile crack is initiated from the inside (Hannant et al., 1973), and is thus not significantly influenced by the surface conditions of the specimen, such as moisture and temperature (Li, 2004; Chen et al., 2012). Since its invention, the test method has continuously been tested to have a low coefficient of variation (Kadlecek et al., 2002). So far, the splitting tension test is considered to be the simplest and most reliable concrete tension test that provides a good estimate of concrete tensile strength.

The relationship between splitting tensile strength and compressive strength cannot be summarized by a constant ratio since splitting tensile strength was observed to decrease with increased compressive strength following a power function relationship (Carino & Lew, 1982). Similar to flexural strength, splitting tensile strength tends to increase with increased rate of loading. Drying of the concrete specimen surface, however, does not affect the splitting tensile strength, in contrast to flexural strength. This is because the specimen surface within the failure plane is subjected to high triaxial compressive stresses (Ozyildirim & Carino, 2006).

2.2.4. Hollow Cylinder Strength Test

An alternative tensile strength test is the hollow cylinder test, which is performed on a hollow cylinder 6 inches in diameter and 12 inches in length (Cantillo & Guzman, 2013). This test method can also be referred to as the ring test when the test is applied on a ring shaped specimen of 6-inch diameter and 1.5-inch thickness (Bremner et al., 1995). The test is very easy to perform. A radial expanding pressure is applied to the inner wall of the hollow cylinder at a prescribed rate until failure of the specimen occurs (Figure 2-9 (a)). This radial pressure is easily generated through the use of an inflatable bladder filled with hydraulic oil positioned within the ring (Figure 2-9 (b)). Tangential tensile stress is applied to the specimen, and the failure pressure can be used to calculate the ring tensile strength of the specimen using stress analysis techniques.

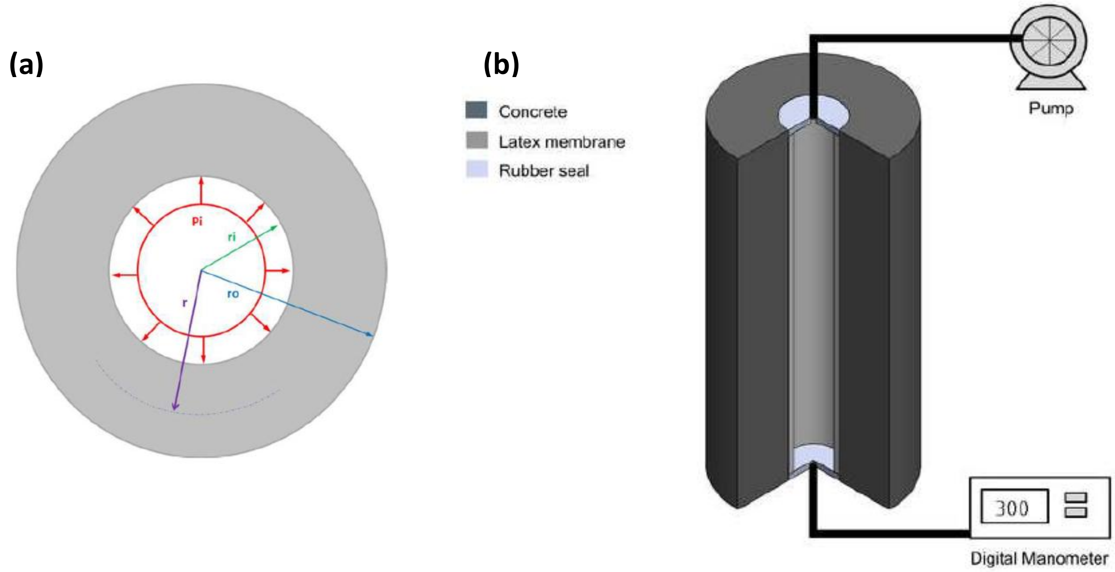


Figure 2-9 - Hollow cylinder test
(a) Geometric Schematics (b) Configuration of Machine (Cantillo & Guzmán, 2013)

Evidence from Cantillo & Guzmán (2013) suggests that the hollow cylinder tensile strength is approximately 17% greater than the splitting tensile strength and is also, on average, 11% that of compressive strength. Failure patterns of the hollow cylinder test are shown in Figure 2-10.

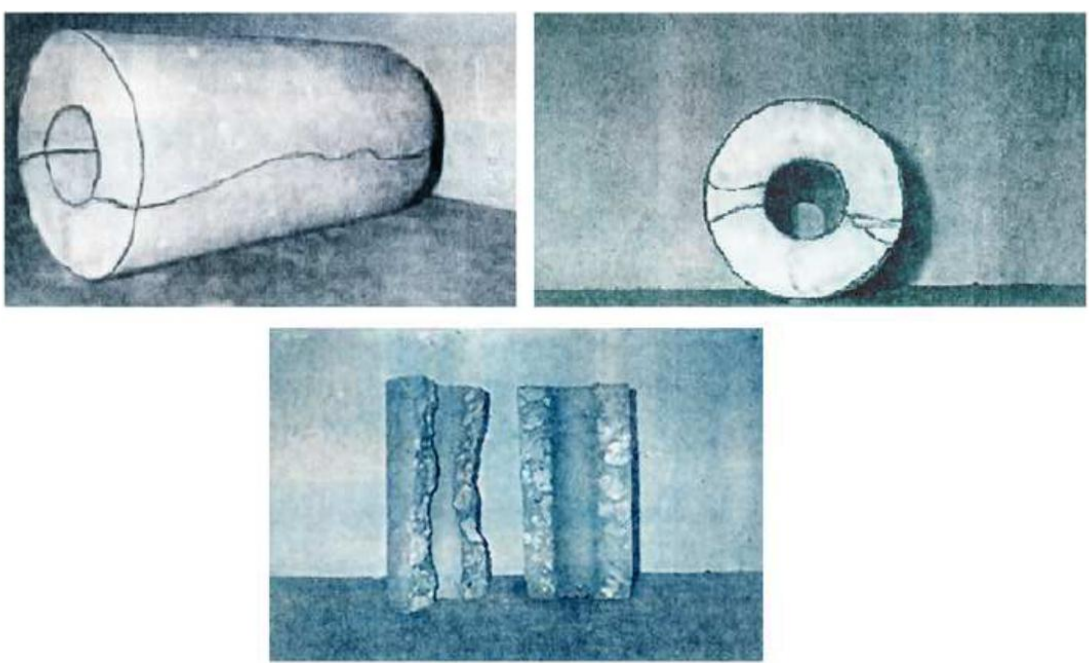


Figure 2-10 - Failure Pattern of the Hollow Cylinder Test
(Cantillo & Guzmán, 2013)

The primary advantage of this technique is the use of specimens similar to that of standard compressive strength test. Also, the uniformly applied pressure on the internal wall guarantees general non-local failure (Cantillo & Guzmán, 2013). This testing method is a relatively new technique, and requires standardization.

2.2.5. Pressure Tension Test

The pressure tension test is also a relatively novel approach for determining the tensile strength of concrete. Clayton (1978) originally developed this indirect tensile test at the Building Research Establishment in the United Kingdom, an experiment in which water and nitrogen gas were tested and compared as the loading media. The pressure tension test also lacks standardization to date.

In the original version of the test, concrete cylinders were placed in an open-ended cylindrical steel pressure chamber. The ends of the cylinders were wrapped in layers of polyvinyl chloride (PVC) tape, which improves the effectiveness of the seal. A rubber O-ring was fixed at each end of the cylinder to act as a seal, creating a cavity between the steel chamber and the cylinder into which a fluid was injected to apply a fluid pressure onto the bare curved surface of the specimen. As the pressure was increased, at a rate of 2.5 MPa per 100 seconds, the specimen eventually failed explosively, in tension, by forming a single failure plane transverse to its axis. The explosive nature of the pressure tension failure was due to the sudden release of considerable amounts of pressure entrapped within the testing chamber when the specimen ruptured. The explosion projected the separated parts of the specimens away from the testing chamber in opposite directions at high velocity. The recorded failure pressure was taken as the tensile strength of the concrete specimen. The theory behind the effect is explained in Section 2.3.

The pressure tension test is a very simple and economical test (Fujikake et al., 2010) that gives reproducible (Bremner et al., 1995) and theoretically reliable results (Do et al., 2001). Unlike other indirect tests, concrete specimens in the pressure tension test are exposed to uniform stress across their entire cross-section and throughout their entire length (Uno et al., 2010). However, the pressure tension strength was observed to be higher than direct tensile strength and splitting tensile strength, and the cause of this is still unknown (Bremner et al., 1995). Just like the splitting tension test, the pressure tension test uses the same concrete specimens as the standard compression test. Sequential tests can be performed within 10 minutes of each other, including preparation, but actual test time will vary depending on the stress rate. Standardization studies are still underway to find the most appropriate stress rate.

2.3. Pressure Tension Effect

In 1912, Percy Bridgman studied the response of solid cylinders subjected to fluid pressure on their external curved surface (Bridgman, 1931). The experiment was performed on cylinder-shaped samples of various materials, but did not include concrete. What he discovered was that although there was no direct force applied along the long axis of the cylinders, all specimens ruptured as if they were being pulled apart by a pure tensile force. Brittle materials, such as glass, ruptured with perfect planes transverse to the axis, while ductile materials like steel exhibited a necking effect before separation, similar to their behaviors under an ordinary tension test.

In the 1970s, Clayton and Grimer (1979) performed the same experiment on concrete using water as the loading fluid (Figure 2-11). The resulting stress required to cause sample failure was less than one-fifth of the crushing strength, which defied the conventional notion that concrete is strong in compression. Similar to how brittle materials reacted in Bridgman's research, the concrete specimen did not fail in a compressive manner, but failed as if it was subjected to an applied tension. The specimen split into two parts with a single plane perpendicular to the axis (Figure 2-12). Moreover, Clayton and Grimer (1979) determined that elongation occurred in the axial direction. The peculiar nature of failure created a paradox, since there was no force acting along the axis perpendicular to the rupture plane. However, Bridgman conceived a theory a few years following his experiment that explained the phenomenon and brought tremendous insight to the understanding of the nature of materials.

(a) (b)

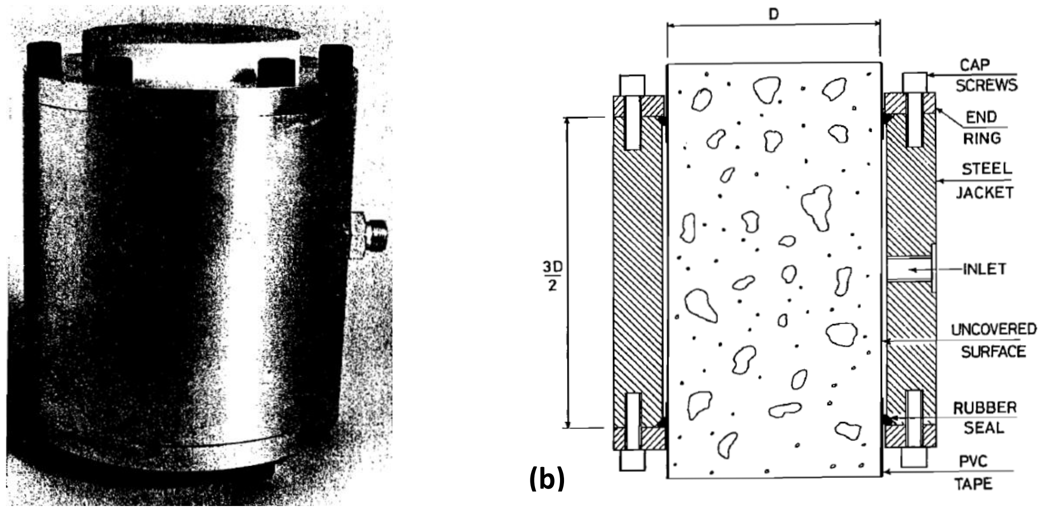


Figure 2-11 - Clayton's Water Pressure Experiment
(a) Photograph of the Apparatus (b) Details of Apparatus (Clayton and Grimer, 1979)

Figure 2-12 - Fracture of Concrete Cylinder due to Water Pressure
(Clayton and Grimer, 1979)

2.3.1. Pressure Tension Mechanics

According to Bridgman's theory, when a cylindrical specimen is subjected to axisymmetrical compressive stress, or when a rectangular prism is subjected to biaxial stress, the resulting effect is equivalent to the addition of a hydrostatic stress, which is an applied triaxial compressive stress, and an axial tensile stress of the same value (Figure 2-13). Considering that the qualitative behavior of the material is not modified by the hydrostatic pressure, this term can be removed from the equation. In other words, the failure mechanism of the material would remain the same regardless of whether the sample is tested in atmospheric pressure or in a high-pressure water environment at the bottom of the sea. Hence, it can be concluded that an axisymmetrical pressure, or biaxial stress, is equivalent to an applied axial tension (Figure 2-13).

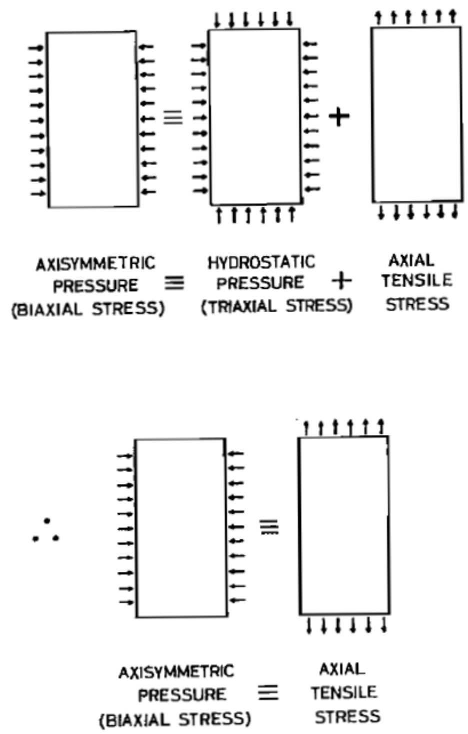


Figure 2-13 - Bridgman's Theory
(Clayton and Grimer, 1979)

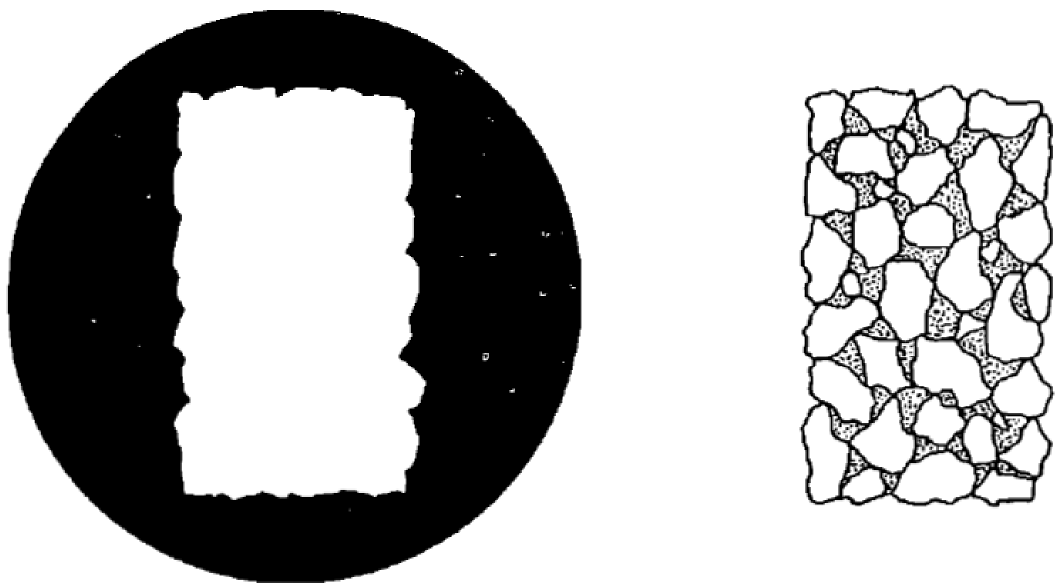
Although Bridgman's theory could be a plausible explanation to the paradoxical phenomenon, his explanation does not explain the source of the tensile stress, as there is no actual tension applied on the cylinder. His mathematical transformation simply changed the reference datum of stress to synthetically produce an applied tensile stress to explain the paradox (Clayton and Grimer, 1979).

Clayton and Grimer (1979) tested Bridgman's theory on concrete cylinders, and found that the hydrostatic pressure stated in the theory has a physical significance. Materials in their natural states are subjected to hydrostatic pressure. For instance, concrete is recognized as being subject to atmospheric pressure. Thus, what is seen as an applied tension is in fact the removal or reduction of a pre-existing external pressure – the axial component of the hydrostatic pressure. This idea was demonstrated in two ways. In the first, a specimen was triaxially compressed at a constant stress using nitrogen gas and simultaneously subjected to increasing direct tension through physical means in the axial direction. In the second case, a concrete cylinder was subjected to a triaxial compression experiment where the rate of increase in applied axisymmetrical stress was set to be higher than the applied axial stress. In both these cases, the mode of cylinder failure was the same as the pressure tension test, and occurred through a fracture transverse to the axis. The stress required for failure was the same as the specimen's uniaxial tensile strength.

2.3.2. Diphase Model of Materials

To further understand the pressure tension mechanism, it is necessary look at the concrete using the diphase approach, which is based on some principles of soil mechanics. Instead of a homogenous system with a single phase, a material consists of two phases: a solid phase and a fluid phase. The solid phase is the agglomeration of all homogeneous particles that provides the strength and stiffness to the material. The scale of the particles considers ranges from the specimen in its entirety, down to the chemical composition, depending on the level of consideration required. The fluid phase is the active constituent that surrounds the solid phase. The main idea of the diphase model is that the solid phase is bound by the fluid phase, which acts as an external pressure (Clayton & Grimer, 1979). The fluid phase is therefore considered to be in a high tensile strain relative to its free state, and this tension exerts a triaxial stress on the solid phase (Grimer & Hewitt, 1969). The rigidity of the solid at a specific level is thus dependent on the level of stress. If this stress is reduced to zero, the material falls apart into its separate components. Consequently, the fluid phase is the source of strength of a material under the diphase model. The solid phase is held together by the fluid phase of the same level.

The diphase model can be expanded in a hierarchical manner (Grimer & Hewitt, 1969), creating different levels of differentiation, or more simply, different levels of consideration. Taking a concrete cylindrical specimen as example, the first hierarchical level can be seen as the specimen itself with its immediate environment. The specimen is the first level solid, and the immediate environment is the first level fluid (Figure 2-14 (a)). The first level constituents can then be divided into second level solids and fluids, which are respectively the particles forming the specimen and their respective surrounding fluid (Figure 2-14 (b)). These solid-fluid systems can be subsequently subdivided down to the atom level.



(a) (b)

Figure 2-14 - The Diphase Model
(a) First Level Solid and Fluid (b) Second Level Solid and Fluid (Clayton and Grimer, 1979)

At every level of consideration, the solid phase is not only affected by the fluid phase of the same level, it is also directly influenced by the fluid phase of the level below, which exerts stress on the separate particles of which the solid phase is composed. The fluid phase of the same level is referred to as the external fluid, while the fluid phase of the level below is referred to as the internal fluid. The two fluid phases interact directly and counteract each other, and the internal fluid reduces the stress induced by the external fluid to reach a stress equilibrium. The difference in stress between the two fluid phases defines the strength of the material, and can be assumed as the absolute stress applied to the solid (Grimer & Hewitt, 1969).

This concept was demonstrated by Osborne Reynolds in 1885 when he filled a rubber bag with loose sand particles. The bag became a rigid body once the air inside was evacuated, which is analogous to the internal fluid being removed, so that the particles are held together as a whole by the atmospheric pressure from the outside. Yet, once the bag is refilled with air, the internal pressure counteracts the external pressure and reduces the absolute stress, the solid bulk formed by the individual particles would therefore collapse, but each particle would remain rigid and strong at their own level, because their individual pressure differences remained unaltered (Clayton and Grimer, 1979).

From the above understanding of material mechanics, 'tension' is an entity that is non-existent as far as the material is concerned (Grimer & Hewitt, 1969). Rather, tension is a manifestation pertaining to the reduction of pre-existing compressive stress. Thus, when the diphase model is applied to concrete, fracture occurs when the compressive stress on the solid is reduced to zero. As per the concept of stress difference, this zero resulting stress is achieved by either decreasing the external stress on the solid phase, or by increasing the internal stress on the solid phase. The pressure tension test is an example of a system that increases the internal stress of the solid. The axisymmetrical fluid pressure applied to the curved surface (Figure 2-15 (a)) permeates the inner pores of the cylinder and increases the internal fluid pressure (Figure 2-15 (b)). Since the change in internal fluid pressure is counteracted by the change of external pressure (Figure 2-15 (c)) in the direction of applied fluid pressure, the stress on the solid is maintained. In the axial direction, on the other hand, the internal fluid pressure is increased until the stress difference is reduced to zero (Figure 2-15 (d)), at which point the specimen fractures. The resulting failure mode of cylinders in the pressure tension test is the same as that under applied axial tension. This is because the failure mechanisms are identical, and involve removing the stress on the solid. As far as the first level solid is concerned, the pressure tension test is identical to a direct tension test, and the pressure tension test can be seen as a valid test of indirect tension.

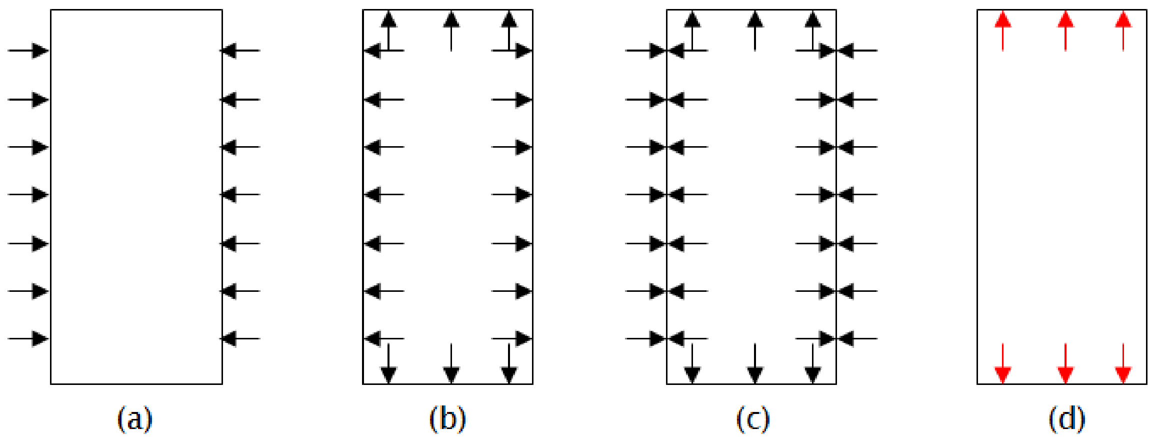


Figure 2-15 – Depiction of Pressure exerted on a Concrete Specimen
(a) External Axisymmetric Pressure (b) Internal Fluid Pressure (c) Counteracting Pressure (d) Resulting Internal Pressure

2.3.3. Loading Medium

The loading medium has a major impact on the accuracy of tests, and has the potential to create a 'tension' effect by increasing the internal fluid pressure on the solid phase, such as in the pressure tension test. The loading medium must be able to effectively transmit the externally applied pressure to the internal phase. Failure to do so results in an increased applied stress required for fracture, as per the concept of stress difference. Therefore, the high apparent strength would not represent the true strength of the specimen. The exhibited strength is proportional to the loading medium's ability to transfer applied stress, and this transfer is enhanced if the loading medium can easily penetrate the inner pores of the specimen.

Richart et al. (1928) studied the change in failure stress on a concrete cylinder subjected to an axisymmetrical load by a close fitting metal sleeve, and found that the resulting failure stress was approximately equal to the crushing strength of the concrete. Later, Langan and Garas (1969) applied the same axisymmetrical load to a concrete cylinder through a wire wound about the curved surface. The fracture occurred at half the crushing strength. Clayton (1978) utilized water to apply this load, and found that failure occurred at twice the applied stress of a direct tension test. Nitrogen gas, on the other hand, gave results in the same range as conventional tensile strength. This is because nitrogen gas was tested to be 5 to 60 times more permeable than water in concrete, depending on the permeability coefficient of the concrete mix (Bamforth, 1987). Therefore, gas transmits pressure much more effectively than water or a rigid body (Clayton, 1978). In all cases, the specimen experienced the same failure mode consisting of a single plane fracture perpendicular to the axis.

In previous versions of the pressure tension test, nitrogen gas was the primary loading fluid because compressed nitrogen was readily available and safe to use. Water is not the favorable loading fluid, for several reasons. Firstly, water has a lower permeability to concrete as opposed to gas fluids. Secondly, the mess created by the water in the explosive failures would result in complications for cleanup and preparation of sequential tests. Lastly, the use of water would change the concrete's original properties, such as water content and pore structure (Li, 2004).

For this research, compressed air was used as the loading fluid using a powerful air compressor. Considering that air, with a density of 1.2929 kg/m^3, is only slightly denser than nitrogen, at 1.2506 kg/m^3, and that nitrogen is the main component of air, the properties of the two fluids are very similar. Pressure tensile results using compressed air in the pressure tension test are expected to be comparable to nitrogen gas.

2.3.4. Seal

All recent variations of the pressure tension setup, including the device used in this research, require the use of tape and O-rings to ensure a tight seal. Uno et al. (2010) investigated the effects of the restraining force acting on the specimen by PVC tape and rubber O-ring. Restraining force due to friction between the materials could potentially bias the results. Despite the fact that polytetrafluoroethylene (PTFE or Teflon) has a lower friction coefficient than PVC, pressure tension test results did not vary between tests using the two materials. Calculations by the investigators also showed that the effect of the restraining force caused by the O-ring is negligible. Therefore, restraining force by both PVC tape and rubber O-rings are not expected to affect the pressure tensile results.

2.3.5. Failure Mechanism

The failure of concrete in tension is initiated through the propagation of cracks. When performed at an adequate rate of applied pressure, the pressure tension test will cause structural failure of the specimen through the path of least resistance. This path generally cuts across the interfacial transition zone (ITZ) in specimens of relatively weaker strength, and across the aggregates when the strength is high (Fujikake et al., 2010).

Fujikake et al. (2010) postulated that when the pressure reaches the specimen's failure strength, a tension crack on the surface is created and propagates through pre-existing microcracks and flaws. This theory was conceived when hollow concrete cylinders having a radial stress of zero on the inner wall surface were observed to fail at the same strength as solid cylinders.

Furthermore, the same researchers observed that the size of pre-existing surficial defect had an impact on the pressure tensile strength. The measured strength was constant up to a critical defect depth, above which the measured value began to drop. This critical depth decreased with greater concrete strength, as would be expected with the increasingly brittle nature of high-strength concrete. The defect width, on the other hand, did not seem to have much effect on the pressure tension results, although their calculations showed that the stress distribution should be affected to a certain degree.

2.3.6. Stress rate

Similar to splitting tensile strength and flexural strength, stress rate also has an effect on the pressure tensile strength of concrete. Pressure tensile strength increases with increased stress rate (Do et al., 2003). A study by Clayton (1978) looked into the effects of water and nitrogen as fluids for the pressure tension test applied at four different rates of stress from 0.02 MPa/s to 20 MPa/s at logarithmic intervals. Both fluids produced fracture planes, the majority of which were within the middle 150 mm of the specimen's length at all stress rates.

The summary plot of the test results is shown in Figure 2-16, with the lines representing the confidence limits. Under low stress rates, at 0.02 MPa/s and 0.2 MPa/s, nitrogen-pressure tension test results were not significantly different, and yielded in the range of the splitting tensile strength. At higher rates of stress, however, results for nitrogen-pressured tests can be linearly correlated with the stress rate, similar to water. Under slow stress rates, microcracks have a greater tendency to propagate and expand, which allows the specimen to fail along the surface with the least resistance. Therefore, the fracture pressure would be constant below a certain threshold stress rate, such as that shown in the nitrogen-pressure tension test in Figure 2-16 between 0.02 MPa/s and 0.2 Mpa/s. This indicates that a range of stress rates can be used for means of standardization and comparison.

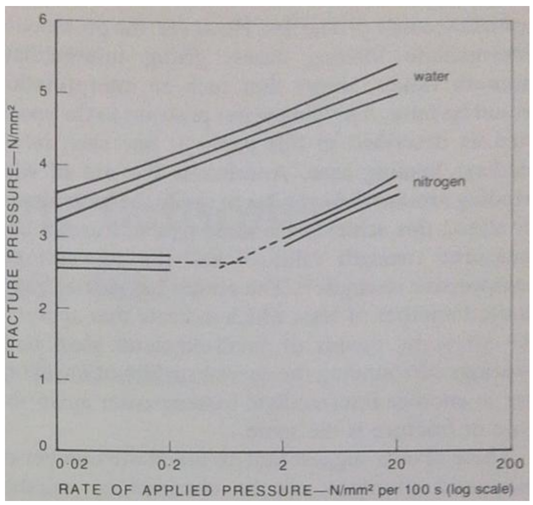


Figure 2-16 – Comparing Water and Nitrogen as Pressure Tension Test Loading Fluid (Clayton, 1978)

2.3.7. Moisture Content

Studies has shown that the pressure tension test is highly sensitive to the moisture content of the specimen (Uno et al., 2010; Li, 2004; Do et al., 2001), but previous research has shown contradicting results in terms of the nature of the relationship between moisture and concrete tensile strength.

In the experiment by Do et al. (2001), air-dried concrete specimens were shown to possess greater pressure tensile strength than water-saturated specimens. In another experiment, by Li (2004), where concrete samples were re-immersed in lime-saturated water after a period of air-drying, it was shown that the pressure tensile strength decreased with increasing period of re-immersion. Yet, the strength of oven-dried concrete was much lower than the strength of both air-dried and re-saturated specimens. However, the rates of applied stress in both of these experiments were controlled manually and non-uniform throughout their testing, which could have introduced experimental error.

In a different study by Uno et al. (2010), where concrete specimens were air-dried after a period of curing, serial tests of concrete strength at various stages of drying exhibited a large decrease in strength once drying commenced, then the strength stayed relatively stable. The authors hypothesized that pressure tensile failure initiates from pre-existing microcracks and flaws on the surface of the specimen. Once the surface is dried and free of moisture, the driving force for the propagation of cleavage cracks induced by the loading fluid becomes much larger because nitrogen gas has a much lower viscosity than water as a loading fluid (Figure 2-17). However, this theory disregards the fact that the specimen had been previously saturated, so the occurrence of an air bubble behind a cleavage crack is highly unlikely.

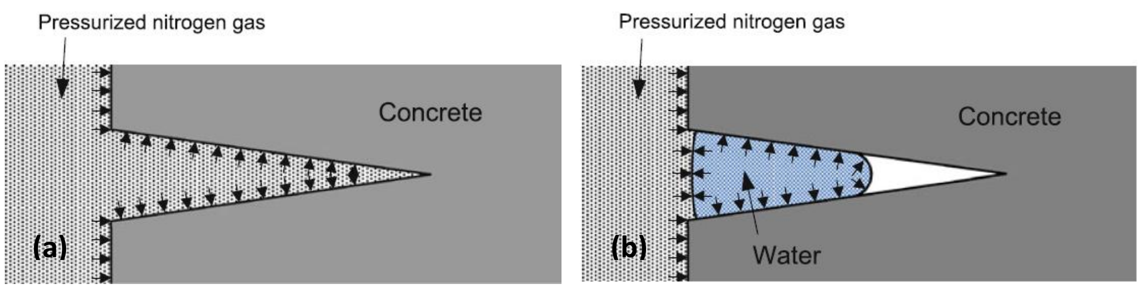


Figure 2-17 - Cleavage Crack Growth
(a) Without Water (b) With Water (Uno et al., 2010)

It is important to note that these previous studies (Uno et al., 2010; Li, 2004; Do et al., 2001) attempted to correlate the relationship between the pressure tensile strength and an indicator of moisture content, but did not directly determine the moisture content of the concrete specimens.

2.3.8. Comparison between Pressure Tension Test and Splitting Tension Test

In contrast to the splitting tension test that concentrates the applied stress on a narrow strip along the longitudinal axis of the test specimen, the pressure tension test subjects almost the entire specimen body to the applied pressure, and may have a higher chance of exploiting the weakest link in the specimen. Based on this assumption, lower tensile strength would be expected to cause structural concrete failure with the pressure tension method. However, the compression zones induced by splitting tension along the outer surface are avoided in the pressure tension. Previous observations have shown that the pressure tensile strength is typically higher than the splitting tensile strength (Bremner et al., 1995; Do et al., 2003; Li, 2004). Pressure tension test results can be as high as three times the splitting tensile strength when the concrete is tested at full saturation (Uno et al., 2010). Moreover, the pressure tension test has been demonstrated to have a smaller variation than the splitting tension test (Fujikake et al., 2010; Do et al., 2003).

2.3.9. Concrete Deterioration Assessment

Another advantageous aspect of the pressure tension test, as opposed to tests which apply an external stress, is that it is far more sensitive to damage from concrete deterioration. The test has been demonstrated to be highly sensitive to changes associated with expansive processes, such as sulphate attack (Hartell et al., 2011; Boyd & Mindess, 2001), alkali-silicate reaction (Bremner et al., 1995), and freeze-thaw damage (Komar and Boyd, 2014). As a result of an internally applied tension stress, pressure tension results reflect the decrease in concrete strength consistent with the duration of exposure to the deteriorating mechanism. The strength loss indicates the level of internal damage of the material. The pressure tension test is capable of detecting such damage with greater success and at a much earlier stage of deterioration than other standard tension test methods, such as the splitting tension test (Komar and Boyd, 2014; Komar et al., 2014).

2.4. Effect of Creep

Creep is defined as the permanent deformation that a material undergoes in time when subject to constant stress. It is a time and stress-dependent deformation that is defined by a strain difference (Atrushi, 2003).

Figure 2-18 depicts the effect of creep on concrete specimens under sustained compressive loading, which is the same effect as that of tension loading. When concrete is placed under a sustained load, creep induces an increase in concrete strain with time, and simultaneously, a reduction in concrete strength. The failure envelope is the range of stress-to-strength ratio within which the tested specimen will fail once the strain exceeds a certain threshold value. The threshold strain limit for a given sustained stress level is termed the failure limit. The failure limit increases with a decrease in stress-to-strength ratio. The failure envelope is separated from the creep envelope by the fracture limit, which is the stress level below which specimens do not fail. The fracture limit is assumed to be approximately 80% of the ultimate concrete strength in compression (Rusch, 1960). The creep envelope is the range of stress-to-strength ratio at which specimens reach maximal strain without failure. The maximum strain that can be experienced by the concrete at a given stress level is called the creep limit. The creep limit decreases with decreasing stress-to-strength ratio. However, these relationships only apply to concrete specimens 24 hours of age or older, due to the time required for strength development in the curing process (Atrushi, 2003; Rusch, 1960).

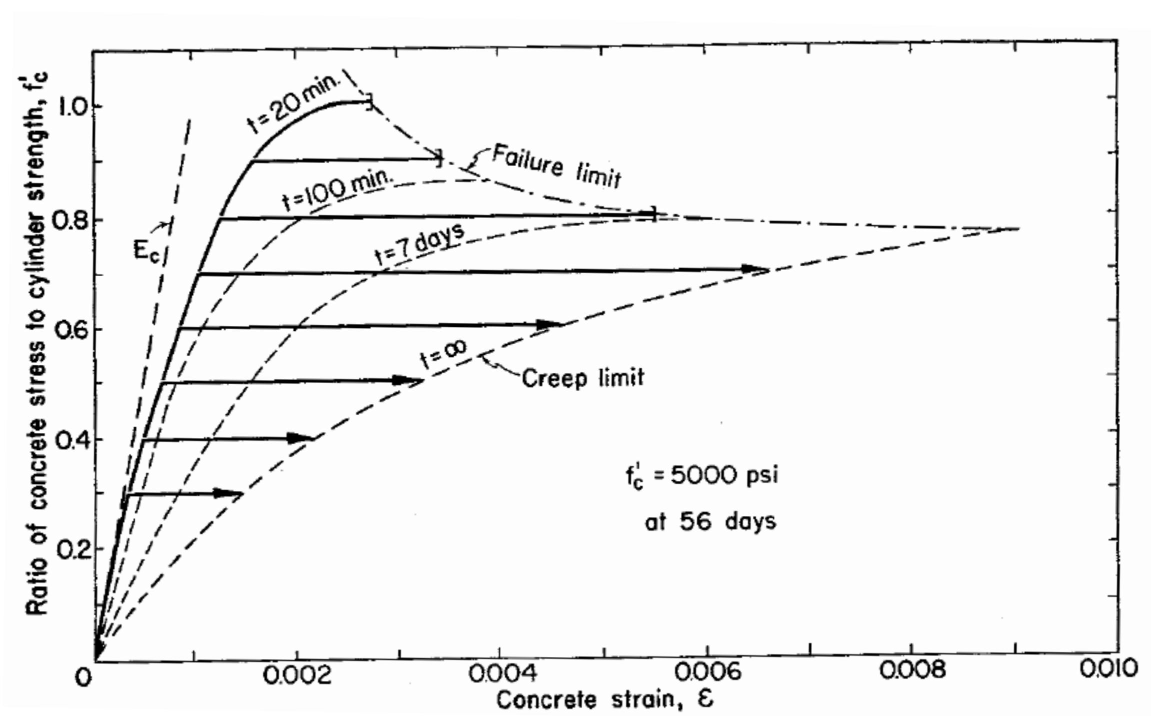


Figure 2-18 - Influence of Load Intensity and Duration on Concrete Strain
(Rusch, 1960)

2.4.1. Effect of Rate of Loading

Like most materials, concrete behavior, is dependent to some degree on the stress rate. Referring to Figure 2-18, the solid line on the left depicts the increase in strain exhibited by the concrete as the stress is increased to the ultimate strength or the sustained load level. Assuming a constant rate of loading, a higher rate of loading would result in a more linear stress-strain curve that would approach the theoretical linear-elastic line as depicted by the dashed line (E_c) on the left. When the applied loading rate is above the empirically determined range of loading rates for a given set of standards specific to the type of strength test, the measured strength is expected to be higher than the actual strength. This is because the fast stress rates do not allow sufficient time for the internal stresses to come to equilibrium. Therefore, cracks are unable to propagate properly across the weakest plane. On the other hand, when the rate of loading is below the stable range, such as the dashed lines for t=100min, t=7days, and t=∞, the specimen would undergo creep, and would fail at a reduced strength with a higher strain as defined by the failure limit.

2.4.2. Tensile Creep

The creep effect of concrete under tension has not been extensively studied because concrete is rarely designed for tensile loading, and compressive creep testing is much easier to perform (Neville et al., 1983; Haroun, 1968). Not only is tensile creep testing difficult to perform, but the process of drying during testing creates concurrent shrinkage that leads to large errors in measurements of creep strain (Haroun, 1968; Rusch, 1960). Creep in tension has been used to estimate the degree of cracking due to shrinkage and thermal stress, and has also been used in the calculation of tensile stress in concrete structures and elements.

The relationship between compressive creep and tensile creep has been studied in the past. Glanville and Thomas (1939) pointed out that that the total creep strain under compression, and the total creep strain under tension are the same at identical stress levels. The group also found that strain increases with time at a decreasing rate (Glanville & Thomas, 1939). This effect was confirmed by the US Bureau of Reclamation (1953) for a stress magnitude of up to one-third of the ultimate tensile strength of concrete. However, there has also been evidence to show that tension creep is higher than compression creep when concrete is dried at 50% relative humidity (Mamillan, 1959). For saturated concrete, a higher creep and a higher rate of creep have been reported (Illston, 1965). Brooks and Neville (1977) observed that the rate of creep in tension for saturated concrete does not decrease over time, unlike in compression.

When subjected to high tensile stress, failure occurs rapidly. When the sustained tensile stress surpasses the fracture limit, a tertiary creep is induced, creating an increasing strain rate on the concrete (Al-Kubaisy & Young, 1975; Domone, 1974). The concrete then experiences a time-dependant failure, even at stress magnitudes lower than the object's actual strength. The fracture limit is defined as 0.85 of 28-day strength for basic creep, 0.75 for saturated concrete (Domone, 1974), and 0.60 for concrete at a relative humidity of 65% (Al-Kubaisy & Young, 1975). The time to failure increases exponentially with decreasing applied stress (Figure 2-19), as in compression. Information on creep at high tensile stresses is of high importance in the estimation of the concrete resistance to cracking.

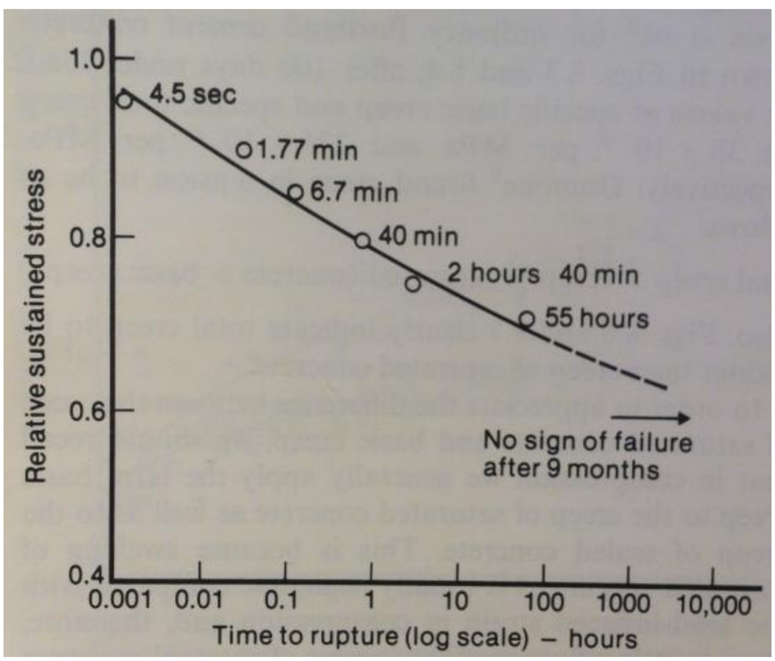


Figure 2-19 - Influence of Sustained Tensile Stress on Time to Rupture for Concrete Drying at 65% Relative Humidity (Al-Kubaisy & Young, 1975)

2.4.2.1. Tensile Creep Testing Methods

Different test methods were conceived to study the tensile creep effect of concrete. Proper tensile creep testing systems need to be able to maintain a constant and uniform stress across the whole cross-section of the specimen while minimizing the amount of maintenance and manual adjustment required (Neville et al., 1983). Due to the same performance difficulties of direct tension tests, tensile creep tests can be performed only with low accuracy (Atrushi, 2003).

Attempts to embed anchorage within the specimens rendered unreliable results due to the induced eccentricity by the anchorage (Swaddiwudhipong et al., 2003). Gluing plates onto the ends with epoxy resin are not effective due to the lack of reliability of tensile strength results, but also does not provide adequate bonding when concrete is damp (Swaddiwudhipong et al., 2003). The test method with bobbin-shaped specimens explained earlier was shown to be a plausible configuration (Neville et al., 1983).

Creep behavior of concrete is strongly affected by many factors, including temperature, specimen age, specimen size, and environmental humidity (Atrushi, 2003). A standard testing method to properly assess the creep effects of concrete in tension has not been developed.

3. EXPERIMENTAL PROGRAM

The goal of this study was to evaluate and define several parameters required for the initial standardization of the pressure tension test on cylindrical concrete specimens. A base set of conditions was chosen as the starting point to explore the range of testing conditions suitable for the new test method. This chapter details the experimental program, including the mixture design, the preparation process of the specimens, the testing procedures, and the materials and equipment used. A total of 473 concrete cylinders were cast and tested in either the compressive strength test or the pressure tension test.

3.1. The Test Regimes

The experimental program for standardizing the pressure tension test was divided into four series:

(1) Evaluation of the variability inherent in the test method

(2) Evaluation of the effects of moisture content in the concrete

(3) Evaluation of the effects of stress rates

(4) Evaluation of the effects of creep

As a means of comparison, each test was performed on three mixture proportions having different water-to-cement ratios (W/C): 0.40, 0.50, and 0.60. Due to capacity restrictions of the available mixer, 12 batches of specimens were cast, three per series corresponding to the three W/C. The batches were numbered as per Table 3-1, and the specimens were named accordingly.

The number of specimens varied for each mixture within each test as a result of extra cylinders cast from each batch. All specimens were molded into the standard cylindrical shape specified by ASTM C470, demolded 24 hours following casting, and cured in limewater until their test date.

Table 3-1 - Batch Numbering

	Batch Number		
W/C	**0.40**	**0.50**	**0.60**
Series 1 – Variability inherent in Test Method	1	2	3
Series 2 – Effects of Moisture Content in the Concrete	4	5	6
Series 3 – Effects of Stress rates	7	8	9
Series 4 – Effects of Creep	10	11	12

A base set of conditions consistent with that of the compression test (ASTM C 39) and the splitting tension test (ASTM C 496) was first chosen in order to explore the basic testing conditions of the pressure tension test. The selected base set specified testing at full saturation and at a constant stress rate of 3 psi/s [0.021 MPa/s]. Based on these base parameters, the testing conditions of each series were modified, one parameter at a time, in accordance to the objectives specified by the name of the series. A summary of the test series is shown in Table 3-2, and the details of each series are described in the following subsections. The labeling of each specimen was based on the respective series and batch number, and is detailed in Appendix D.

Table 3-2 - Summary of the Experimental Program

	Quantity of Specimens		**3**	**5**	**5**	**5**	**5**	**5**	**5**	**Other Loading Conditions**
1	**Variability inherent in the Test Method**	0.40 W/C								Fully Saturated 3 psi/s [0.021 MPa/s]
		0.50 W/C								
		0.60 W/C								
						Number of days of drying				
2	**Effects of Moisture Content in the Concrete**	0.40 W/C		Day 0	Day 1	Day 2	Day 3	……	Day X	3 psi/s [0.021 MPa/s] Moisture content measured with largest recovered debris
		0.50 W/C		Day 0	Day 1	Day 2	Day 3	……	Day X	
		0.60 W/C		Day 0	Day 1	Day 2	Day 3	……	Day X	
						Stress Rate*				
3	**Effects of Stress Rate**	0.40 W/C		1 psi/s	2 psi/s	3 psi/s	4 psi/s	5 psi/s		Fully Saturated
		0.50 W/C		1 psi/s	2 psi/s	3 psi/s	4 psi/s	5 psi/s		
		0.60 W/C		1 psi/s	2 psi/s	3 psi/s	4 psi/s	5 psi/s		
						Stress Level (% of Ultimate Stress)				
4	**Effects of Creep**	0.40 W/C	Ultimate	86%	88%	90%	92%	94%		Fully Saturated 3 psi/s [0.021 MPa/s]
		0.50 W/C	Ultimate	86%	88%	90%	92%	94%		
		0.60 W/C	Ultimate	86%	88%	90%	92%	94%		

*Unit conversion: 1 psi/s = 0.007 MPa/s; 2 psi/s = 0.014 MPa/s; 3 psi/s = 0.021 MPa/s; 4 psi/s = 0.028 MPa/s; 5 psi/s = 0.034 MPa/s

3.1.1. Series 1 – Variability inherent in Test Method

The first series aimed to evaluate the inherent variability in the pressure tension test. The concrete specimens underwent the pressure tension test at a stress rate of 3 psi/s [0.021 MPa/s] until failure. A total of 25 specimens per batch were tested for 0.40 and 0.50 W/C, and 26 specimens for 0.60 W/C. All specimens of each batch were tested at the same age (± 1 day) in a saturated condition.

3.1.2. Series 2 – Effects of Moisture Content in the Concrete

The second series had the goal of evaluating the extent to which the specimen's moisture content affects the pressure tension results, and to specify the moisture condition of the concrete for standardization of the pressure tension test. Similar studies have been performed in the past on the effect of concrete moisture conditions, but neither the moisture content nor the relative moisture content have ever been explicitly studied in relation to pressure tensile strength.

On the first day of testing for each specified batch, all specimens of the batch were removed from the curing bin and left to dry at ambient room temperature. A set of 5 specimens were tested as they were removed from the water on the first day of testing, Day 0, to represent full saturation conditions. Five specimens were tested on each consecutive day. The tests were performed on 45 specimens of 0.40 W/C, 46 specimens of 0.50 W/C, and 51 specimens of 0.60 W/C. The testing spanned a period of 9 days for 0.40 and 0.50 W/C, and 10 days for 0.60 W/C, and each specimen underwent the test at a stress rate of 3 psi/s [0.021 MPa/s].

Due to the explosive nature of the pressure tension test, only the largest recoverable portion of the specimen was used to measure the moisture content. The recovered portion of the specimen was weighed and dried in an oven for a period of 5 to 7 days. When the weight of the concrete sample stabilized, the sample was reweighed and recorded. The difference in weight before and after drying was used to calculate the moisture content of the specimen.

3.1.3. Series 3 – Effects of Stress Rates

The third series studied the effects of stress rate on the pressure tensile strength of concrete. These results were used to determine the proper range of stress for the standardization of the test method. Five stress rates were used, varying from 1 psi/s [0.007 MPa/s] to 5 psi/s [0.034 MPa/s], at 1 psi/s [0.007 MPa/s] increments. A minimum of five specimens per batch were tested with each stress rate. A total of 25 specimens were tested per batch for 0.40 and 0.60 W/C, and 27 specimens for 0.50 W/C. All specimens of each batch were tested at the same age (± 1 day) and in saturated condition.

3.1.4. Series 4 – Effects of Creep

The fourth series was a study of whether creep would affect the pressure tension results obtained from the base testing conditions. Strain measuring equipment was not available for this analysis, so the time to failure was used as a proxy to study creep effects, assuming that all specimens from the same batch would strain at the same rate. Three specimens per batch were first used to determine the ultimate tensile strength of the batch. From that average strength, five stress levels ranging from 86% to 94% of ultimate strength, at 2% increments, were subsequently calculated. Specimens were then loaded at a stress rate of 3 psi/s [0.021 MPa/s] until the specified stress level was reached, and held at the stress level until failure occurred. Due to the time constraints of this study, the test time for this series was limited to 40 minutes. The results of the specimens that did not fail within this time limit were still included in the analysis. This series of tests also attempted to explore the possibility of performing tensile creep testing with the pressure tension test, and to map out the relationship between the stress level and the time to failure under sustained stress.

Five specimens per W/C were tested at each stress level. A total of 28 specimens per batch were tested for all W/C. All specimens of the same batch were tested at the same age (± 1 day) and in the saturated condition.

3.2. Specimen Preparation

The specimen preparation process in this experimental program included formulating the mixture proportions, preparing the mixture ingredients, as well as concrete sample mixing, casting, and curing. Because the properties of the materials were not all known during the specimen preparation process, standard test procedures were used to determine the relevant properties.

3.2.1. Materials

3.2.1.1. Portland Cement

The Portland cement used in all concrete mixtures was general use hydraulic cement (Type GU), as designated by Canadian specification CAN/CSA-A3001-03, provided by Lafarge Canada's cement plant in St-Constant. Type GU cement is equivalent to Type I cement as detailed in ASTM C 150/C150M. This cement is a common or general purpose cement and it is used when special properties, such as sulphate resistance or increased early strength, are not required.

3.2.1.2. Aggregates

The fine aggregate was a natural silica sand supplied by Lafarge Canada from the St-Gabriel-de-Brandon quarry. The coarse aggregate was dolomite of class 5-14 mm provided by Bauval. All aggregates were dry when used. Tests conforming to ASTM C127, ASTM C128, ASTM C136 and CAN/CSA-A23.1 were performed to obtain the properties of the aggregates. The results are presented in Table 3-3. Detailed results are presented in Appendix A and B. Both types of aggregates were tested and found to meet ASTM C33 requirements for grading and quality of fine and coarse aggregate for use in concrete.

Table 3-3 - Aggregate Properties

Properties	Fine Aggregate	Coarse Aggregate
Type	Silica	Dolomite
Class		5-14mm
Oven-dry Relative Density	2.439	2.714
Percent Absorption	1.19%	0.59%
Oven-dry Rodded Bulk Density		1567.44 kg/m^3
Fineness modulus	2.74	

3.2.1.3. Water

The water used for all concrete mixtures was potable tap water taken directly from the city supply of Montreal, Canada, consistent with Portland Cement Association (PCA) guidelines which state that any odorless drinkable water can be used as mixing water for concrete making (Kosmatka et al., 2002a).

3.2.1.4. Admixtures

A superplasticizer, Grace Construction Products ADVA 140M, was used in all concrete mixtures in order to increase the mixture's fluidity and workability during mixing and casting of the concrete specimens. 500mL was added per 100kg of cement. No other admixtures were used.

3.2.2. Mix Design

Using the aggregate properties described in Subsection 3.2.1.2, three different concrete mixtures were formulated following the PCA guidelines (Kosmatka et al., 2002b). The three mixtures had different W/C: 0.40, 0.50, and 0.60. Since the aggregates were dry when used for mixing, the amount of water was adjusted as per the design guidelines to take into account the water absorption by the aggregates. The final mixture proportions are shown in Table 3-4. A detailed breakdown of mixture design can be found in Appendix C. The quantities per cast varied depending on the target number of concrete cylinders for each test.

Table 3-4 - Mixture Proportions

Content	0.40 W/C	0.50 W/C	0.60 W/C
Water	195.2 kg/m3	196.6 kg/m3	197.6 kg/m3
Cement	453.6 kg/m3	362.9 kg/m3	302.4 kg/m3
Coarse	903.1 kg/m3	903.1 kg/m3	903.1 kg/m3
Fine	897.3 kg/m3	980.2 kg/m3	1035.4 kg/m3
Super Plasticizer	2.3 L/m3	1.8 L/m3	1.5 L/m3
TOTAL DENSITY	**2451.5 kg/m3**	**2444.6 kg/m3**	**2440.0 kg/m3**

3.2.3. Mixing and Casting Procedure

The mixing and casting of the 473 specimens took place at McGill University in the Department of Civil Engineering and Applied Mechanics concrete lab from May to September 2012 over a cumulative period of 8 days.

The mixing and casting procedures were carried out following the ASTM C172 and ASTM C192 standards. Mixing was performed with a Crown Construction Equipment S-6SR 6 cubic feet towable mortar/plaster mixer, modified to include a spiral mixing blade designed for concrete mixing. All of the coarse aggregate was introduced into the mixer along with half of the mixing water and half of the superplasticizer prior to the start of the mixer. The mixer was then turned on to run for a few rotations. The fine aggregate, Portland cement and the remaining half of the mixing water and superplasticizer were then added in the aforementioned sequence with the mixer running. Once all ingredients were added, the batch was mixed for 3 minutes, followed by a 3-minute rest, and then mixed for another 2 minutes.

The slump of each batch was measured immediately after mixing as per ASTM C143. The results are presented in Table 3-5. The high slump level was expected due to the use of superplasticizer.

Table 3-5 - Slump Results

	Batch	W/C	Slump (mm)
Series 1 – Variability inherent in Test Method	1	0.40	192
	2	0.50	208
	3	0.60	205
Series 2 – Effects of Moisture Content in the Concrete	4	0.40	222
	5	0.50	246
	6	0.60	208
Series 3 – Effects of Stress rates	7	0.40	207
	8	0.50	212
	9	0.60	155
Series 4 – Effects of Creep	10	0.40	207
	11	0.50	212
	12	0.60	169

All specimens were cast in two layers of equal thickness and vibrated using a vibrating table in order to achieve proper consolidation of the concrete. Fresh concrete was poured into moulds that conformed to the ASTM C 470 with dimensions of 101.6 mm x 203.2 mm (4" x 8") and 100 mm x 200 mm. Both sizes of moulds were used due to the limited quantity of each size available for casting. The minute difference in diameter is tolerated by ASTM C 470. In the case of the pressure tension tests, a difference in diameter of 1.6 mm can also be neglected because the pressure is applied along the lateral surface of the cylinders. The difference in length is irrelevant because only a limited portion of the cylinder body is subjected to the axisymmetrical load. Furthermore, due to irregularities of natural concrete and the possible deformation of plastic moulds during curing, small differences between cast cylinders are expected.

3.2.4. Curing

Following casting, all specimens were covered with durable impervious plastic to prevent evaporation of water from unhardened concrete. After 24 hours of initial curing, the specimens were removed from their moulds, and placed into a limewater curing bin until their test date.

3.3. Pressure Tension Testing Apparatus

The pressure tension apparatus used for this research was constructed at McGill University by the Civil Engineering Materials Laboratory. Minor modifications and adjustments of the apparatus were performed throughout the course of this experimental program to improve operator safety, but the core components of the apparatus remained unaltered.

This version of the pressure tension instrument had two major parts: the physical components, and a computer software controlling the whole test. The loading fluid was compressed air instead of pure nitrogen gas due to the availability of an air compressor. The air compressor allowed the loading fluid to be quickly refilled when depleted, and allowed continuous testing without excessive delay.

3.3.1. Physical Components

The pressure tension apparatus was similar to its predecessors in its mechanical aspect, with the exception of the air compressor, a multitude of security features, and computer stress control. Figure 3-1 shows the test setup, which consists of:

a) an air tank that supplies the pressure for the testing;
b) an air compressor used to recharge the air tank periodically;
c) a flow regulator valve that controls the maximum allowable pressure from the air tank for testing;
d) a computer controlled actuator valve that controls the stress rate during testing;
e) a pressure transducer (not shown in figure) that monitors the chamber pressure and communicate this information to the computer;
f) a battery powered pressure sensor to monitor the line pressure in case of power failures and to verify the electronic pressure indicators in case of technical malfunction;
g) a data acquisition unit (not shown in figure) that converts digital signals between the pressure transducer, the actuator, and the computer;
h) a testing chamber where specimens are loaded;
i) a safety valve permitting the release of chamber pressure and prevents pressure buildup when the machine is not in use;

j) multiple line cutoff valves between the air tank and the testing chamber for safety concerns;
k) a computer that runs the test software.

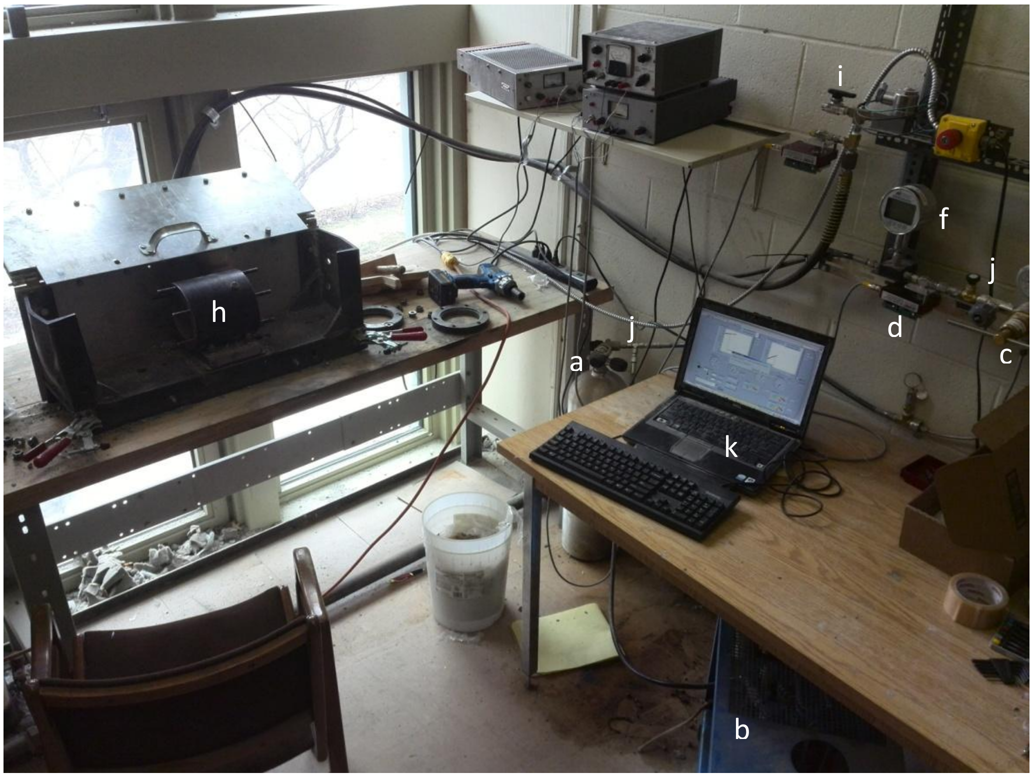


Figure 3-1 - Pressure Tension Test Setup

The testing chamber was a custom fabricated hollow cylindrical tube made of steel that allowed a 101.6 mm x 203.2 mm (4" x 8") cylindrical concrete specimen to be placed inside (Figure 3-2). It had three components: two hollow rings with an interior corner bevel and a central chamber. When the concrete specimen was properly placed into the central chamber, the two rings were tightened onto either end of the chamber with 6 threaded screws (Figure 3-3). An O-ring with an interior diameter of 101.6 mm (4") and thickness of 6.35 mm (0.25") was placed on the cylindrical surface of the concrete specimens at the ring bevels (Figure 3-2) to provide a positive seal of the gas inside the testing chamber. The chamber was welded onto an enclosing steel frame to prevent flying debris when specimen failure occurred.

Figure 3-2 - Testing Chamber

Figure 3-3 - Steel Ring Fitted onto the Central Chamber

3.3.2. Software

The present version of the pressure tension test is controlled by software created using National Instruments LabView. The software controls the stress rate, and records the chamber pressure during testing. It also permits the operator to automatically close pressure valves and override the test when necessary, for safety concerns. A picture of the software interface is shown in Figure 3-4.

During testing, the software continuously runs through a control loop at 0.1 second intervals. At every iteration, the pressure transducer in the testing chamber takes a pressure reading and compares this to the reference pressure calculated by the software based on the control parameters. Basic control parameters are entered prior to the start of a test, but they can also be changed while the software is running. Based on the difference between the chamber pressure and the calculated reference value, the software communicates with the actuator valve to increase or decrease the pressure. The software is capable of accounting for its errors in every iteration and thus increase precision of applied pressure as the test is running.

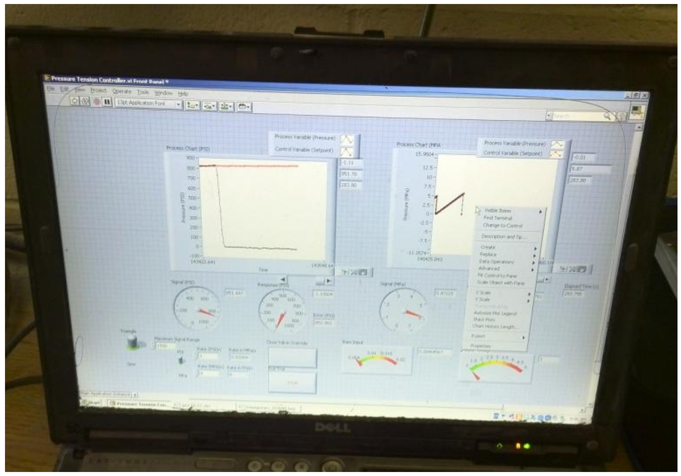


Figure 3-4 - Pressure Tension Test Software Interface

3.4. Specimen Testing

All specimens cast for this research were subject to either the compressive strength test or the pressure tensile strength test.

3.4.1. Compression Test

A total of 6 specimens per batch underwent the compression test as per the ASTM C 39 standard to determine the average compressive strength of the batch. Three specimens per batch were subjected to the compression test following 28 days of limewater curing, and three others were tested on the first day of the pressure tension testing. Prior to compressive testing, both ends of the specimens were ground. The diameters of each specimen were measured using a caliper. All compressive tests were performed with an MTS uniaxial hydraulic loading system.

3.4.2. Pressure Tension Test

With the exception of the specimens in Series 2 for evaluating moisture content effects, all specimens were removed from the curing water 5 to 10 minutes prior to testing for preparation purposes. Specimens for Series 2 were allowed to air dry as detailed in Subsection 3.1.2 until test day.

Before being placed into the testing chamber, both ends of the each specimen were wrapped with a layer of 50.8 mm (2") wide, 0.05 mm (0.002") thick PVC tape. PVC tape was used to ensure a smooth surface beneath the O-rings, and served to tighten the fit as well as prevent leaks due to irregularities in the concrete surface. The excess water on the surface of the concrete cylinders was allowed to partially dry off and removed with a towel to allow proper application of the PVC tape.

When ready, specimens were placed into the testing chamber. Although PVC tape was used to create a uniform surface, due to the elastic nature of the tape, holes larger than half the thickness of the O-rings still posed a risk of creating a passage for leak. Therefore, concrete specimens were carefully placed in the testing chamber so that no major holes were located beneath the O-rings. O-rings were subsequently placed around the test specimen (Figure 3-5), followed by the hollow steel rings that seal the pressure chamber. O-rings can be damaged during the explosive failures of specimens (Figure 3-6), especially with specimens of high strength. Therefore, periodic replacement of damaged O-rings was required. Leaks can result in inadequate system response, causing a sudden large increase in applied pressure, which would produce an inaccurate representation of the specimen strength.

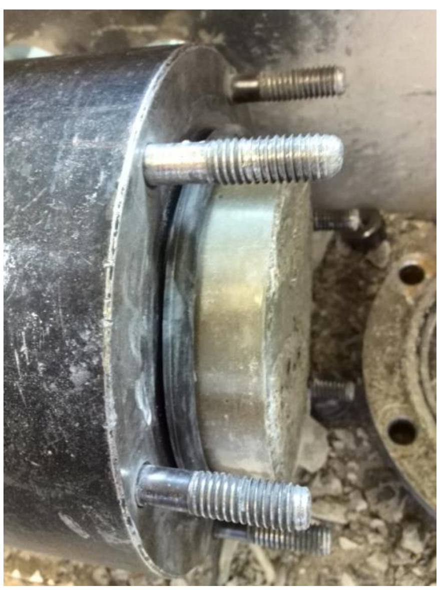


Figure 3-5 - O-Ring Properly placed on Test Specimen

Figure 3-6 - O-Ring damaged after Test

Loading parameters as detailed in Section 3.1 were entered into the software. After completion of security protocols, such as ensuring the protective cover was properly closed and the surrounding area cleared, the test was initiated by the operator using the software. The specimens were then subjected to their respective loading conditions until failure occurred.

4. RESULTS AND DISCUSSION

The results of the experimental program are presented and discussed in this chapter. Additional observations are also included. The raw pressure tension results for all test specimens can be found in Appendix D.

4.1. Pre-investigation Results

Each tested batch consisted of a number of specimens, as listed in Table 4-1, that underwent the pressure tension test at full saturation and with a stress rate of 3 psi/s [0.021 MPa/s]. The average pressure tensile strengths (f_t) from these specimens were compared with respect to their average compressive strengths (f_c) in Figure 4-1, Figure 4-2, and Table 4-1. The batch age on test day was also studied.

The batches were tested at different ages, with the youngest batch being 71 days old (Figure 4-1). Therefore, none of the specimens were tested prematurely, before the age of 28 days. However, due to the difference in age between specimens within the same series, the test results using different W/C could not be directly compared. Moreover, strength is expected to increase with an increase in concrete age (Bungey et al., 2006), but this trend was not displayed in Figure 4-2 since the tested specimens came from different batches. The non-conformity in the trend can be justified because each batch was prepared separately and independently.

For all batches, compressive strength tests were performed at the same age as the pressure tensile strength tests. With the amount of specimens tested in the same condition, it is possible to confirm that the pressure-tensile-strength-to-compressive-strength ratio (f_t/f_c) is consistent with that of other tensile strength tests. This ratio was determined to range from 0.10 to 0.16. It was observed that the f_t/f_c increased with lower f_c, except in the case of Batch 4 and 5, as shown in Table 4-1.

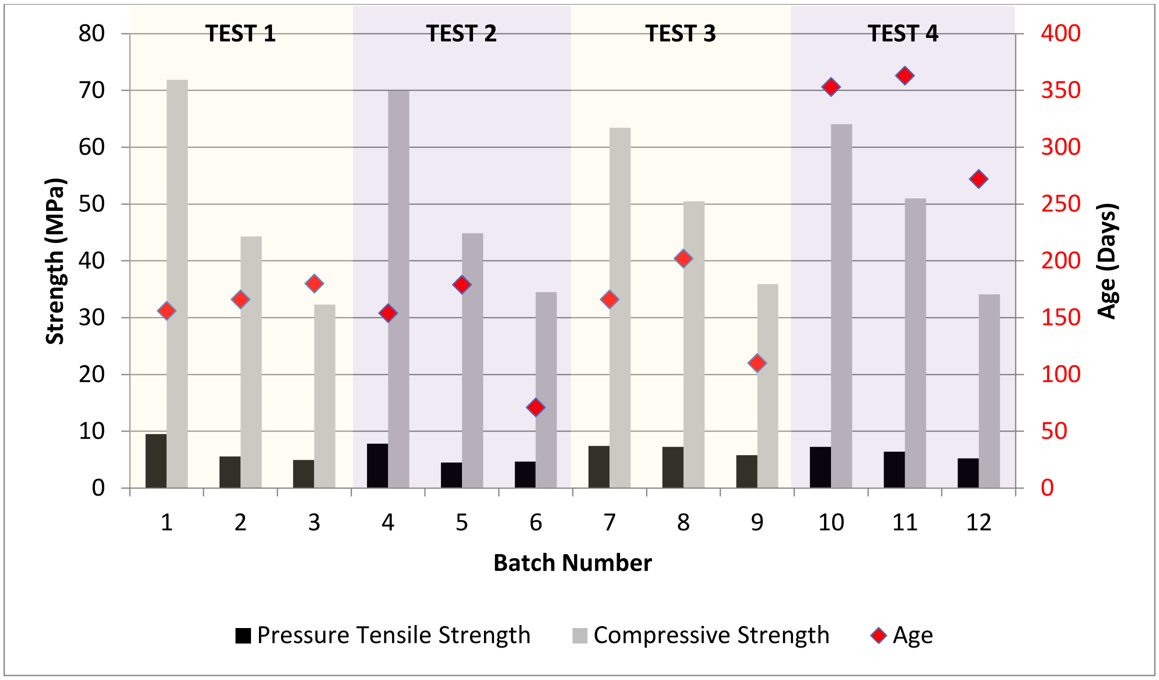


Figure 4-1 - Concrete Test Age, Compressive Strength, and Pressure Tensile Strength (Grouped by Batch)

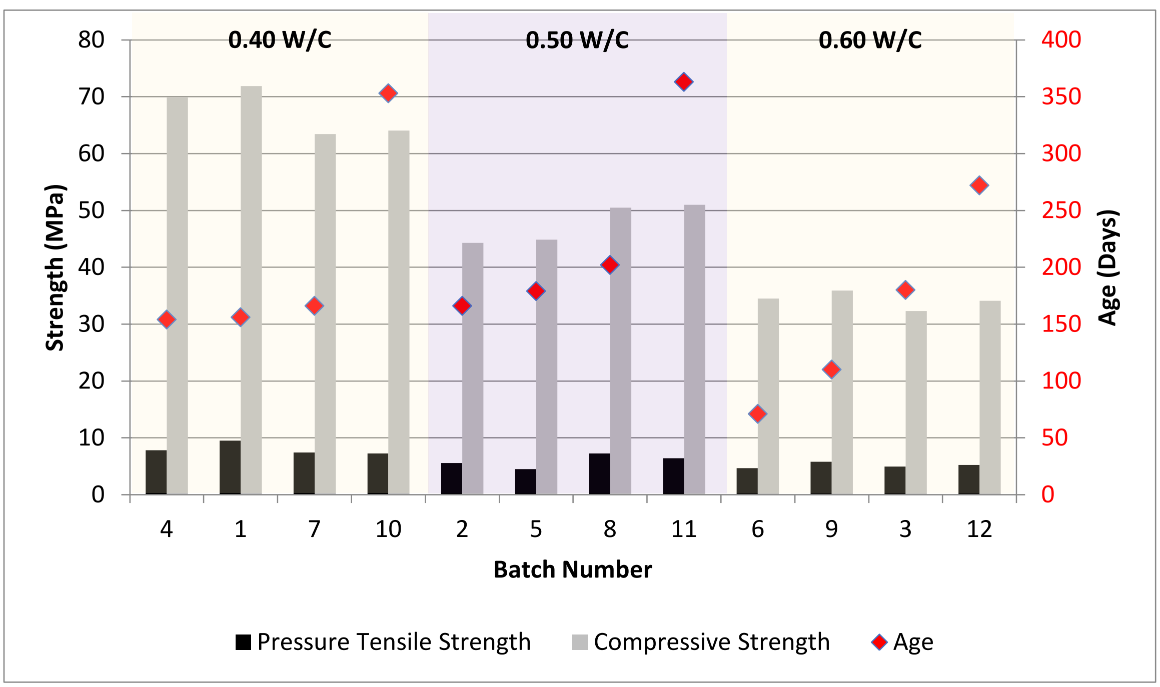


Figure 4-2 - Concrete Test Age, Compressive Strength, and Pressure Tensile Strength (Grouped by W/C and Age)

Table 4-1 - Concrete Test Age, Compressive Strength, and Pressure Tensile Strength

Test Series	1 – Variability in Test Method			2 – Effects of Moisture Content			3 – Effect of Stress Rate			4 – Effects of Creep		
Batch #	1	2	3	4	5	6	7	8	9	10	11	12
W/C	0.4	0.5	0.6	0.4	0.5	0.6	0.4	0.5	0.6	0.4	0.5	0.6
Age (Days)	**156**	**166**	**180**	**154**	**179**	**71**	**166**	**202**	**110**	**353**	**363**	**272**
# in tension	*25*	*25*	*26*	*5*	*5*	*5*	*5*	*4*	*5*	*7*	*9*	*5*
f_t (MPa)	9.48	5.56	4.93	7.83	4.47	4.69	7.40	7.27	5.79	7.26	6.42	5.24
# in compression	*3*	*3*	*3*	*3*	*3*	*3*	*3*	*3*	*3*	*3*	*3*	*3*
f_c (MPa)	71.88	44.31	32.32	69.87	44.85	34.51	63.43	50.50	35.92	64.02	51.00	34.10
f_t/f_c	**0.13**	**0.13**	**0.15**	**0.11**	**0.10**	**0.14**	**0.12**	**0.14**	**0.16**	**0.11**	**0.13**	**0.15**

4.2. Variability inherent in Test Method

This series of test showed that the variability in the pressure tension test method is low. The results of the tests are presented in Figure 4-3 in the same order as their testing sequence. Visually, the results for both 0.50 W/C and 0.60 W/C were relatively consistent for all specimens. For 0.40 W/C, however, specimens #21 to #23 gave results that were unusually high compared to the other results of the same batch. Reviewing the original test data showed that these results were outliers. The cause for the high values was likely due to variability within the concrete batch, and not technical issues with the testing apparatus.

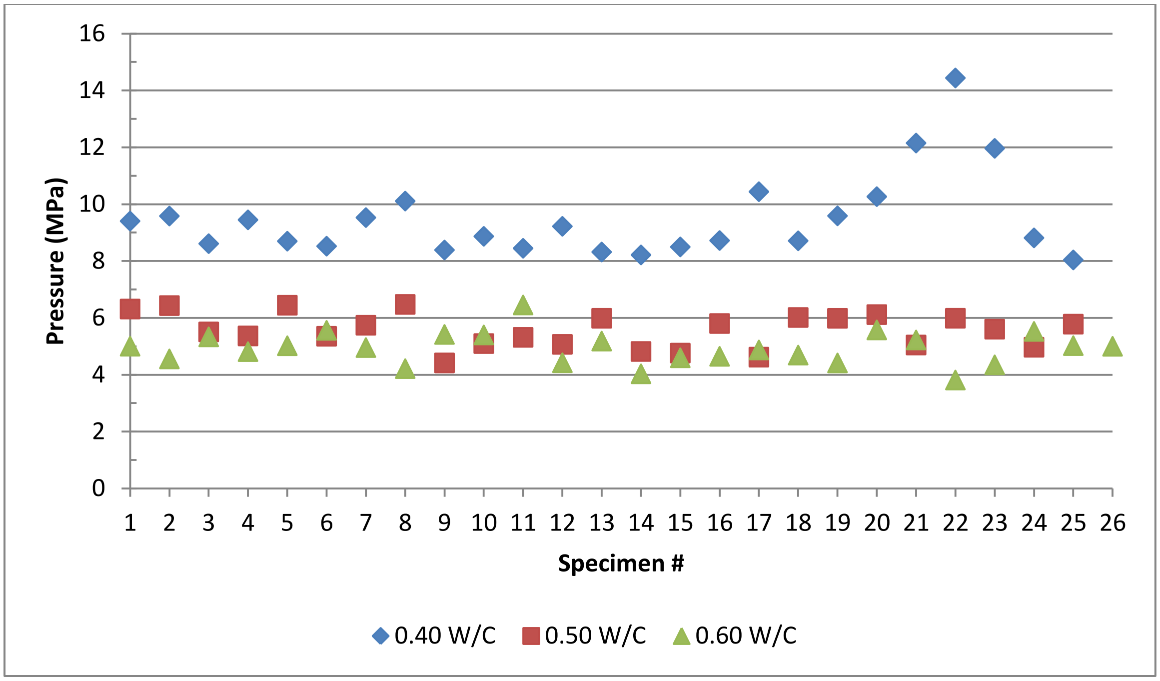


Figure 4-3 - Variability inherent in Test Method – Results of all Specimens

The average pressure tensile strength of concrete specimens at each W/C are presented in Table 4-2 and Figure 4-4. Although tested at different ages, the three batches were tested at relatively close dates. Therefore a trend line was created to observe the relationship. The trend line reflects the typical strength-to-W/C curve as described by Abrams (1918). The standard deviation is higher with a lower W/C. This is expected because the effect of individual microstructure variation is more pronounced with lower W/C concrete (Pann et al., 2003). The coefficient of variation varied between 10.86% and 15.60%, which is highly comparable to the splitting tension test.

Table 4-2 – Variability inherent in Test Method – Averaged Results

W/C	0.40		0.50		0.60	
	MPa	psi	MPa	psi	MPa	psi
Average Strength	9.479	1374.8	5.559	806.3	4.927	714.5
Standard Deviation	1.478	21.42	0.604	87.54	0.570	82.67
Coefficient of Variation	15.60%		10.86%		11.57%	
Number of Samples	25		25		26	

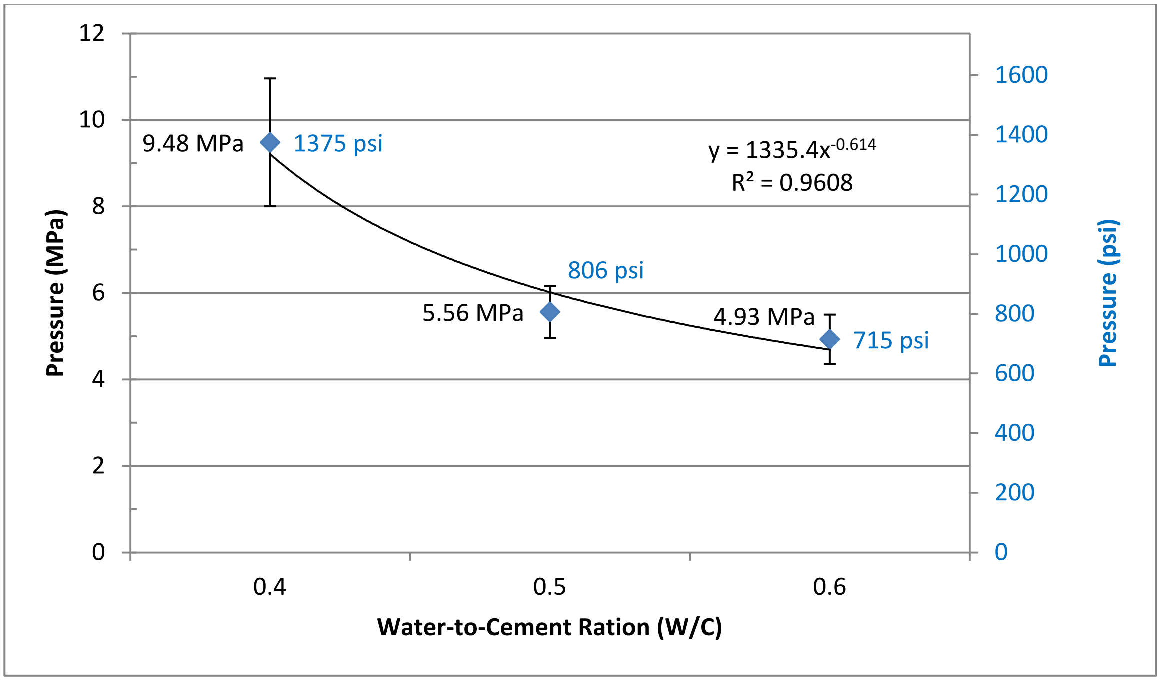


Figure 4-4 – Variability inherent in Test Method – Averaged Results

4.3. Effects of Moisture Content in the Concrete

Like all batches, the specimens for this series were limewater-cured until their test date. Therefore, for each batch in this series, the five specimens that were tested at the age of 0 days were fully saturated. The average value of their measured moisture contents was set to represent the 100% relative moisture content. The relative moisture content of each specimen was then compared with the pressure tensile strength. The original results are presented in Figure 4-5, Figure 4-7, and Figure 4-8 for 0.40 W/C, 0.50 W/C, and 0.60 W/C, respectively. The black data points represent results of fully saturated specimens that were tested on day 0.

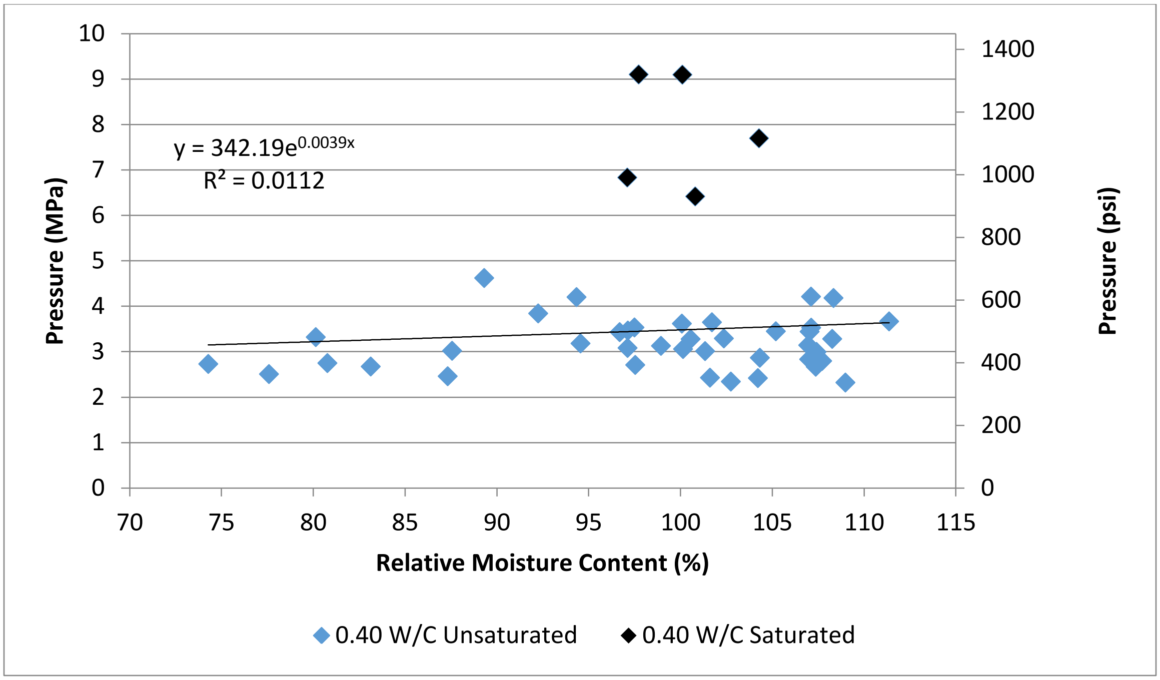


Figure 4-5 – Effect of Moisture Content in the Concrete – Results of 0.40 W/C

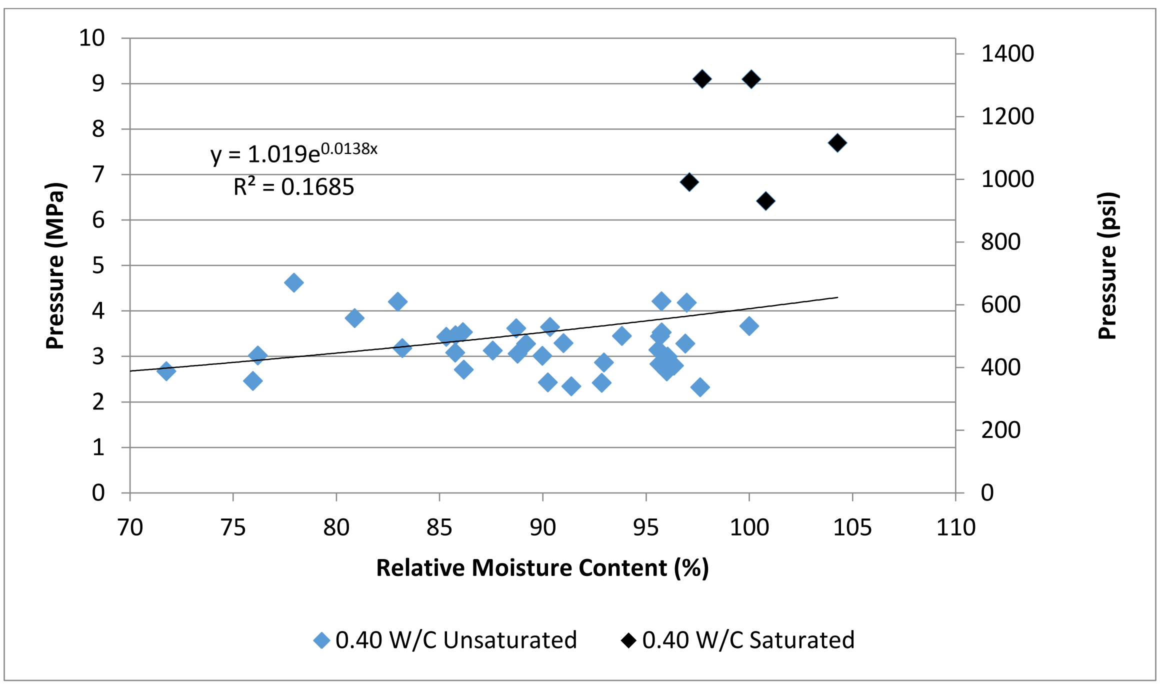


Figure 4-6 - Effect of Moisture Content in the Concrete – Normalized Results of 0.40 W/C

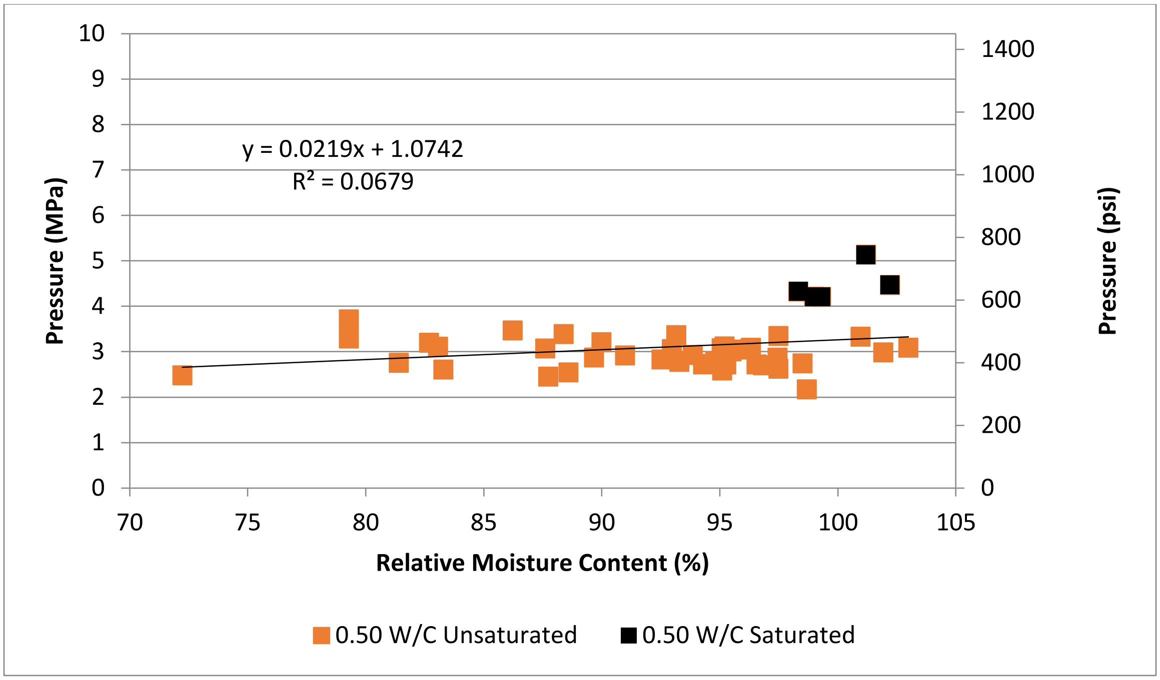


Figure 4-7 - Effect of Moisture Content in the Concrete – Results of 0.50 W/C

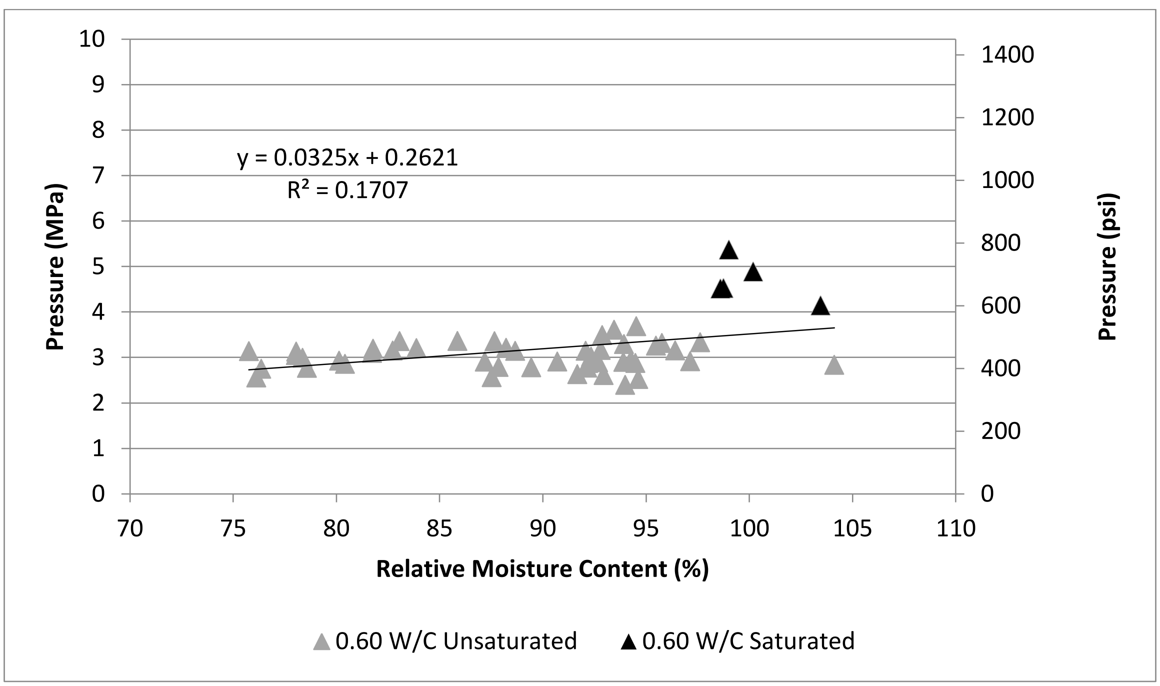


Figure 4-8 - Effect of Moisture Content in the Concrete – Results of 0.60 W/C

Specimens subjected to a period of drying were expected to have lower moisture contents. However, as can be seen in Figure 4-5, approximately half the specimens of 0.40 W/C had measured relative moisture contents exceeding 100%, which was determined based on the average values of the fully saturated specimens. The measured moisture content exceeded the saturation limit by as much as 11%. This effect only occurred with 3 specimens in 0.50 W/C with a maximum of 3% moisture excess, and 1 specimen in 0.60 W/C exceeding by 4%. Results of dried 0.40 W/C specimens were normalized with respect to the largest measured moisture content. The normalized results are presented in Figure 4-6. The apparent 'oversaturation' in the measured relative moisture content of the air-dried specimens implies that either an experimental error was made when measuring the moisture content, or that a loss of moisture with the fully saturated specimens occurred during testing. The method for moisture content measurement was the same for all specimens, and the fact that this effect mainly occurred with one specific batch, 0.40 W/C, means that human error was not likely the main cause. Therefore, there must have been a loss of moisture.

The moisture content of all tests of this series was measured after the specimen was tested. A plausible explanation for the loss of moisture is that, since concrete is a porous medium, a portion of the enclosed humidity was carried out by pressurized air escaping the system through the unsealed portion of the specimens, such as the ends of the concrete cylinders. On occasions, a singing sound was heard during tests, which could have been due to the air leaving the system. From a physics standpoint, higher pressure would induce a greater airflow and a stronger driving force moving the water out of the specimen. Therefore, this effect is more pronounced with specimens subjected to higher pressure, such as 0.40 W/C. This reasoning suggests that there is not enough accuracy in any of the measured moisture contents, regardless of the W/C, to establish a correlation between moisture content and the pressure tensile strength. Moisture content is therefore excluded in the following discussion.

As can be observed in all W/C, fully saturated specimens gave pressure tensile results that are much higher than the drier specimens. Dried specimens experienced a strength decrease of as much as 70.3% for 0.40 W/C, 51.4% for 0.50 W/C, and 48.8% for 0.60 W/C. A greater decrease in strength is observed with a lower W/C. The strength comparison is presented in Table 4-3, the results of all specimens subjected to drying are grouped as unsaturated specimens as a whole. Regardless of W/C and number of drying days, the strength stabilized within a small range after an initial loss of moisture. The coefficient of variation for unsaturated specimens varied between 9.6% and 17.5%. A graphical representation of this stability is shown in Figure 4-9, Figure 4-10, and Figure 4-11 for 0.40 W/C, 0.50 W/C, and 0.60 W/C, respectively. These graphs regroup the results with respect to the number of drying days. Since the diffusion rate of concrete is fairly slow (Chen et al., 2012), the immediate decrease in strength following the drying process indicates that the surficial moisture is potentially the main contributing factor for the change in strength. These results confirm the findings by Uno et al. (2010), as described in Subsection 2.3.7. However, since their proposed theory is still questionable, further research is required to understand the mechanism behind the loss of strength with an initial loss of moisture. The degree to which the unsaturated specimens are stabilized in strength can only be evaluated with an accurate measure of moisture content.

Table 4-3 - Effect of Moisture Content in the Concrete – Average Change in Strength

W/C	Saturated Specimens			Unsaturated Specimens			Strength Decrease		
	Avg Strg (MPa)	SD	COV	Avg Strg (MPa)	SD	COV	Max.	Min.	Avg.
0.40	7.83	1.25	15.9%	3.18	0.56	17.5%	70.3%	41.0%	59.4%
0.50	4.47	0.39	8.70%	2.94	0.31	10.7%	51.4%	16.9%	34.2%
0.60	4.69	0.46	9.90%	3.02	0.29	9.60%	48.8%	21.3%	35.6%

Note: SD = Standard Deviation; COV = Coefficient of Variation

The average relative moisture content is also plotted in Figure 4-9 to Figure 4-11. It must be noted, however, that the moisture content of specimens tested on the same day can vary largely. As shown in Table 4-4, although the coefficient of variation lied between 1% and 4% on most days of testing, it was as high as 17.3% for specimens of day 4 of 0.40 W/C, 7.07% for specimens of day 1 of 0.50 W/C, and 9.57% for specimens of day 5 for 0.60 W/C. Individual microstructural variations play a major role in the rate of drying of specimens (Pann et al., 2003; Haroun, 1968). Therefore, a more rigorous or a different testing strategy is necessary to accurately measure the moisture content in the pressure tension test, and to properly study how moisture affects the pressure tensile strength.

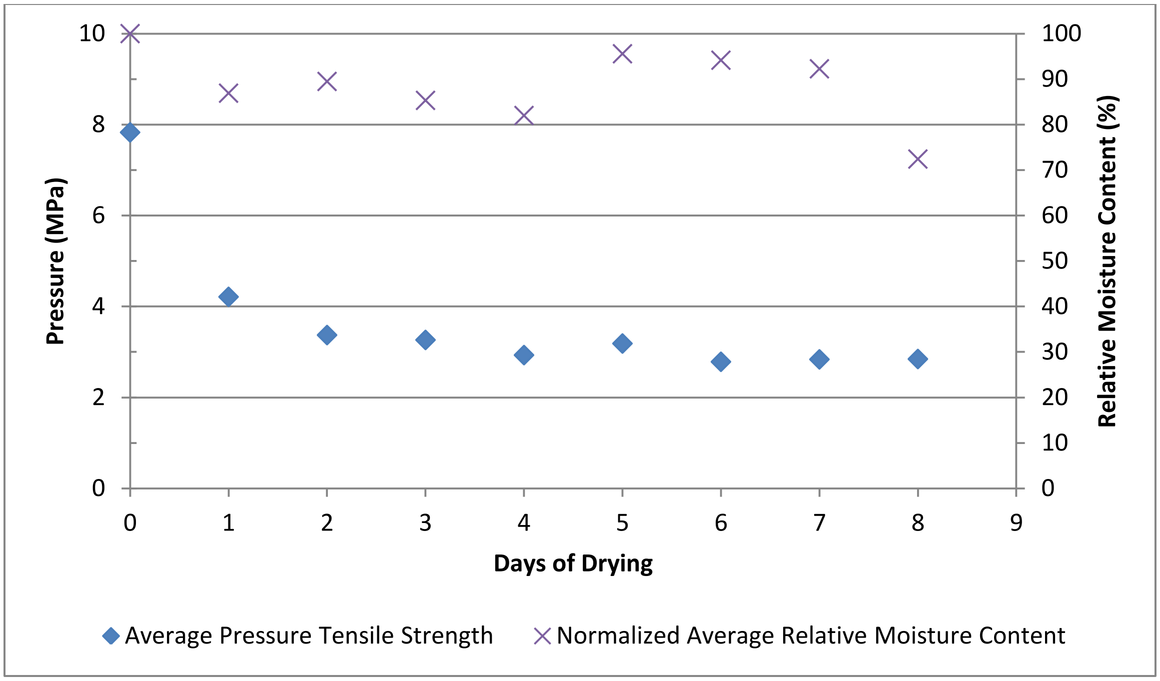


Figure 4-9 - Effect of Moisture Content in the Concrete – Average Normalized Results of 0.40 W/C in terms of Days of Drying

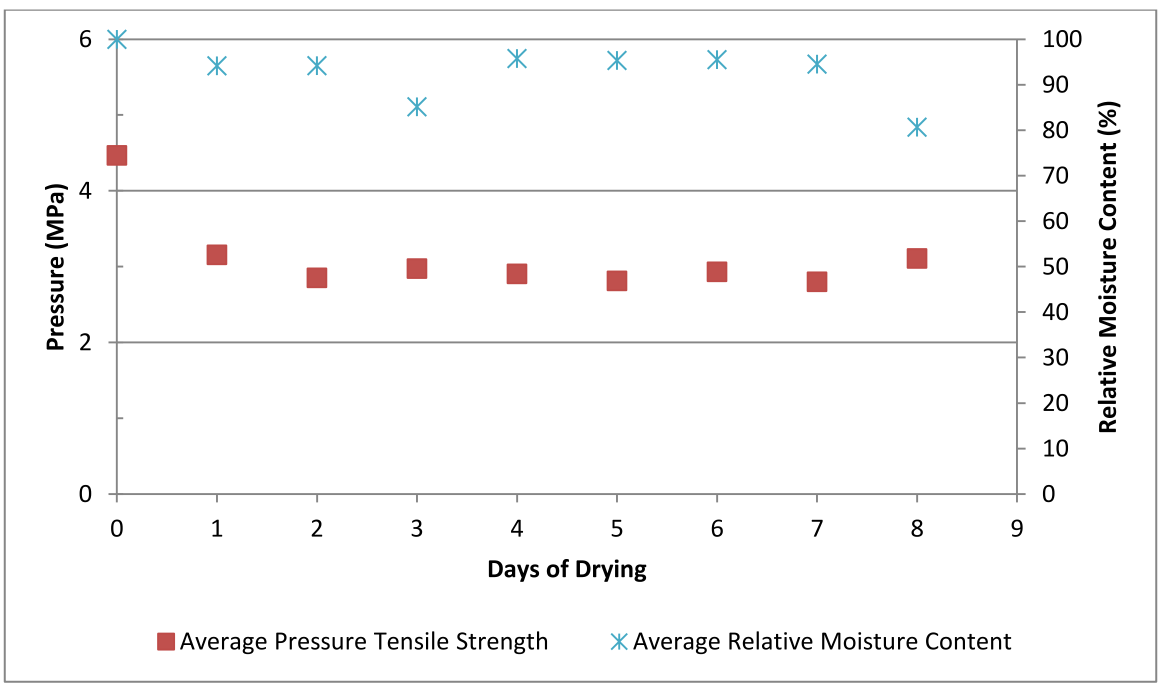


Figure 4-10 - Effect of Moisture Content in the Concrete – Average Results of 0.50 W/C in terms of Days of Drying

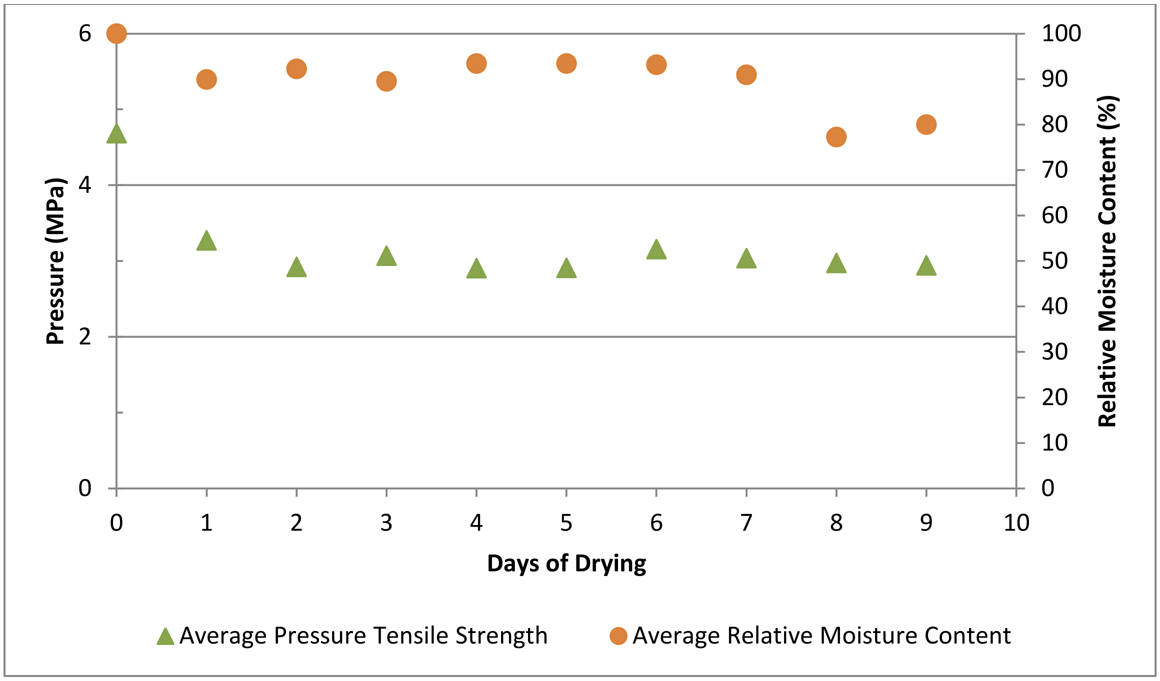


Figure 4-11 - Effect of Moisture Content in the Concrete – Average Results of 0.60 W/C in terms of Days of Drying

Table 4-4 - Effect of Moisture Content in the Concrete – Coefficient of Variation of Moisture Content in terms of Days of Drying

Days of Drying	0	1	2	3	4	5	6	7	8	9
0.40 W/C	2.85%	8.98%	2.92%	1.28%	17.3%	4.43%	2.43%	2.27%	4.20%	
0.50 W/C	1.63%	7.07%	3.55%	6.99%	5.92%	2.79%	2.17%	1.21%	6.61%	
0.60 W/C	2.03%	5.14%	1.85%	9.57%	3.81%	3.77%	3.85%	4.46%	1.58%	2.85%

4.4. Effects of Stress Rates

Following the pressure tension tests in this series, statistical analyses were performed to determine the variations between test results at different stress rates. The pressure tension results for the stress rates of 1 psi/s [0.007 MPa/s] to 5 psi/s [0.034 MPa/s] are presented in Figure 4-12, and the average results for each W/C are presented in Figure 4-13, Figure 4-14, and Figure 4-15. One specimen from 0.40 W/C, two from 0.50 W/C, and three from 0.60 W/C were discarded because the system experienced leak and failed with a sudden increase in loading pressure.

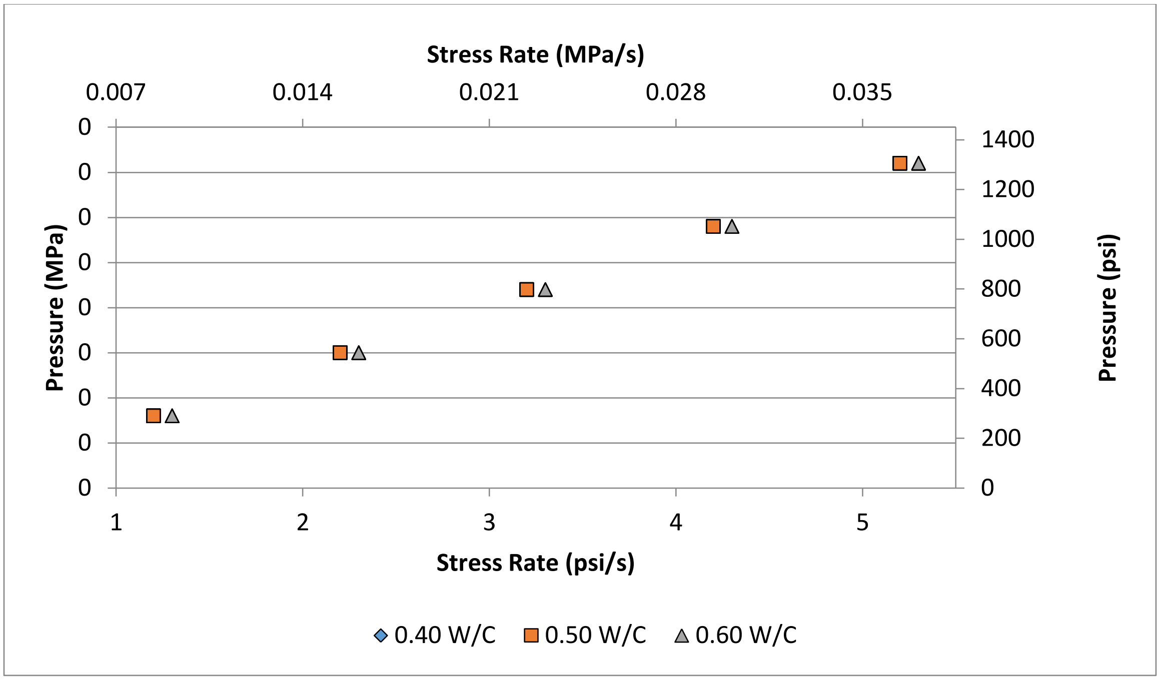


Figure 4-12 - Effect of Stress Rates – Comparison of all Results

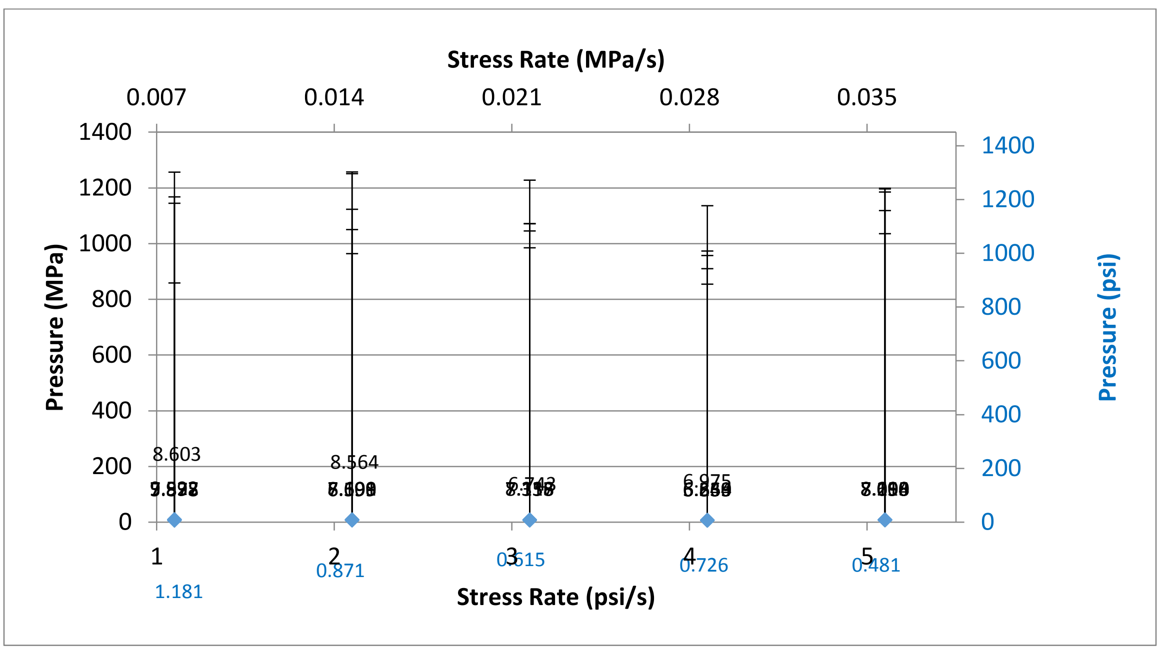


Figure 4-13 - Effect of Stress Rates – Results of 0.40 W/C

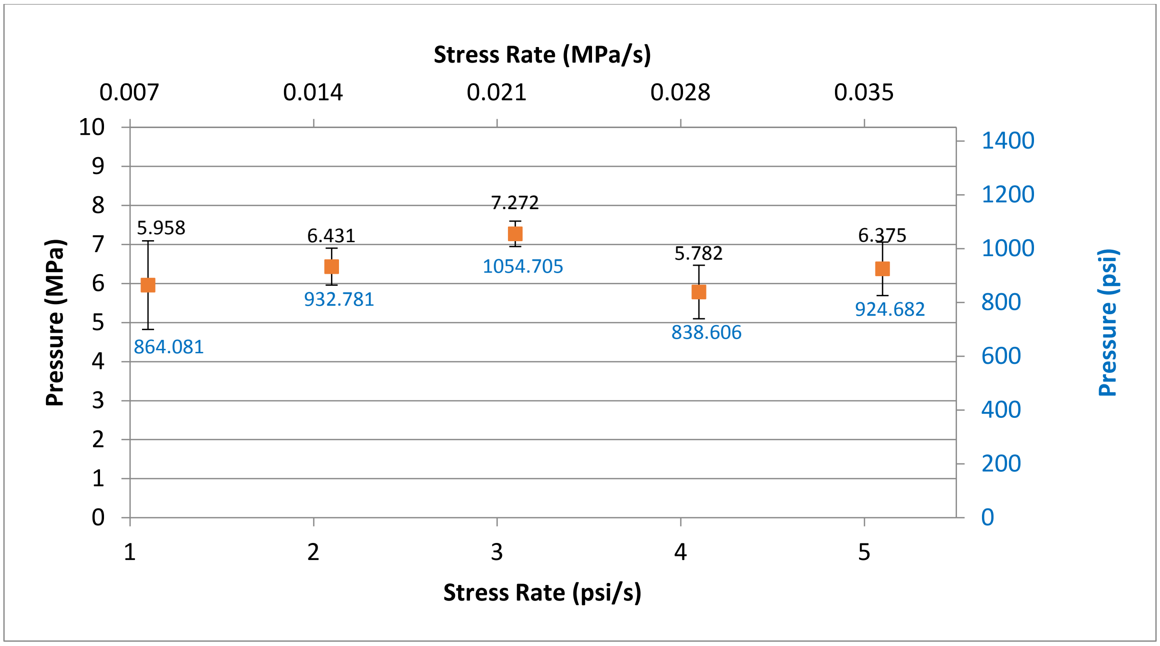


Figure 4-14 - Effect of Stress Rates – Results of 0.50 W/C

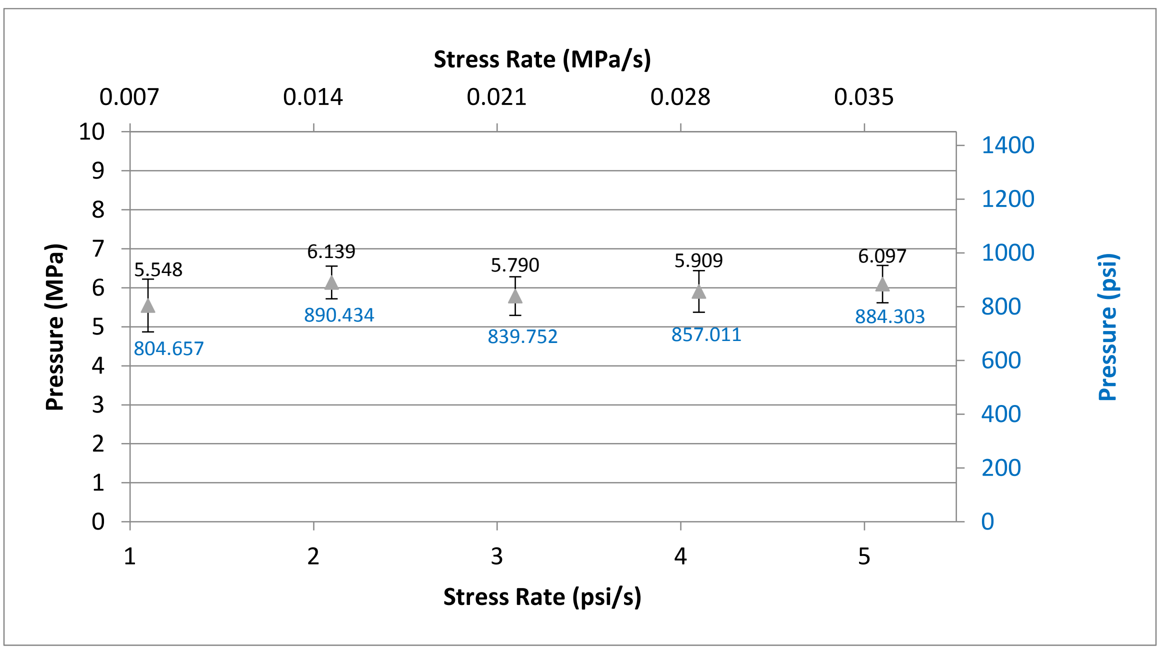


Figure 4-15 - Effect of Stress Rates – Results of 0.60 W/C

Analysis of variance (ANOVA) was first used to assess the differences between the stress rates for each W/C in Table 4-5, Table 4-6, and

Table 4-7. There was no statistical difference in test results between the different stress rates for 0.40 W/C and 0.60 W/C. However, at least one of the five stress rate results for 0.50 W/C were different from the others.

Table 4-5 - Effect of Stress Rates – ANOVA for 0.40 W/C Results

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
1 psi/s [0.007 MPa/s]	4	30.31292	7.57823	1.393619
2 psi/s [0.014 MPa/s]	5	38.65221	7.730442	0.759454
3 psi/s [0.021 MPa/s]	5	36.97784	7.395568	0.378557
4 psi/s [0.028 MPa/s]	5	33.07834	6.615668	0.526623
5 psi/s [0.034 MPa/s]	5	39.25706	7.851412	0.231608

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	4.748695	4	1.187174	**1.917103**	0.148968	**2.895107**
Within Groups	11.76582	19	0.619254			
Total	16.51452	23				

Table 4-6 - Effect of Stress Rates – ANOVA for 0.50 W/C Results

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
1 psi/s [0.007 MPa/s]	4	23.83053	5.957633	1.288016
2 psi/s [0.014 MPa/s]	5	32.15649	6.431298	0.22409
3 psi/s [0.021 MPa/s]	4	29.08776	7.27194	0.106333
4 psi/s [0.028 MPa/s]	6	34.69191	5.781985	0.469765
5 psi/s [0.034 MPa/s]	6	38.25274	6.375457	0.470218

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	5.967134	4	1.491784	**3.050894**	0.040884	**2.866081**
Within Groups	9.77932	20	0.488966			
Total	15.74645	24				

Table 4-7 - Effect of Stress Rates – ANOVA for 0.60 W/C Results

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
1 psi/s [0.007 MPa/s]	4	22.19167	5.547918	0.457992
2 psi/s [0.014 MPa/s]	3	18.41798	6.139327	0.175675
3 psi/s [0.021 MPa/s]	5	28.94943	5.789886	0.245098
4 psi/s [0.028 MPa/s]	5	29.54441	5.908882	0.284457
5 psi/s [0.034 MPa/s]	5	30.48528	6.097056	0.229206

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	0.920827	4	0.230207	**0.822103**	0.52889	**2.964708**
Within Groups	4.76037	17	0.280022			
Total	5.681197	21				

A two-tailed Student's T-test was therefore performed between each individual group within a given W/C to further assess the differences at 95% confidence interval, assuming equal variance. The p-values for each comparison are presented in Table 4-8, Table 4-9, and Table 4-10. According to the statistical results, it can be confirmed that the stress rates made no measurable difference for 0.60 W/C, and for most of 0.40 W/C. At 0.40 W/C, the results between 4 psi/s [0.028 MPa/s] and 5 psi/s [0.034 MPa/s] show that they are statistically unlikely to be of the same group. However, when comparing either group with the remaining groups, as well as to the batch as a whole, the results show that all groups are the same. With 0.50 W/C, on the other hand, the results of 3 psi/s [0.021 MPa/s] are significantly different from those of other stress rates. Yet, considering that the group is located in the center of the range of stress rates being studied, and that the two other W/C are statistically determined to be stable within this range, it is assumed that the results of this group are due to variations within the batch.

According to the statistical assessment, the studied range of stress rate, 1 psi/s [0.007 MPa/s] to 5 psi/s [0.034 MPa/s], gives no difference in pressure tensile strength. Hence, repeatability is ensured at these stress rates. Nonetheless, more work is required to confirm these findings.

The rates that were not covered in this research should also be examined to determine the complete range of stress rate that gives consistent results.

Table 4-8 - Effect of Stress Rates – Student T-Test for 0.40 W/C Results

0.40 W/C		p-value				
	Stress Rates	**1 psi/s [0.007 MPa/s]**	**2 psi/s [0.014 MPa/s]**	**3 psi/s [0.021 MPa/s]**	**4 psi/s [0.028 MPa/s]**	**5 psi/s [0.034 MPa/s]**
Compared to	**1 psi/s [0.007 MPa/s]**		0.82965	0.77152	0.17382	0.6481
	2 psi/s [0.014 MPa/s]	0.82965		0.50266	0.05918	0.79265
	3 psi/s [0.021 MPa/s]	0.77152	0.50266		0.10411	0.22808
	4 psi/s [0.028 MPa/s]	0.17382	0.05918	0.10411		0.01312
	5 psi/s [0.034 MPa/s]	0.6481	0.79265	0.22808	0.01312	
	All results	0.75809	0.47636	0.93573	0.05676	0.29365

Table 4-9 - Effect of Stress Rates – Student T-Test for 0.50 W/C Results

0.50 W/C		p-value				
	Stress Rates	**1 psi/s [0.007 MPa/s]**	**2 psi/s [0.014 MPa/s]**	**3 psi/s [0.021 MPa/s]**	**4 psi/s [0.028 MPa/s]**	**5 psi/s [0.034 MPa/s]**
Compared to	**1 psi/s [0.007 MPa/s]**		0.42007	0.06767	0.76503	0.48384
	2 psi/s [0.014 MPa/s]	0.42007		0.01976	0.10751	0.88085
	3 psi/s [0.021 MPa/s]	0.06767	0.01976		0.00396	0.04296
	4 psi/s [0.028 MPa/s]	0.76503	0.10751	0.00396		0.16454
	5 psi/s [0.034 MPa/s]	0.48384	0.88085	0.04296	0.16454	
	All results	0.4358	0.77151	0.03009	0.14419	0.88009

Table 4-10 - Effect of Stress Rates – Student T-Test for 0.60 W/C Results

0.60 W/C		p-value				
	Stress Rates	**1 psi/s [0.007 MPa/s]**	**2 psi/s [0.014 MPa/s]**	**3 psi/s [0.021 MPa/s]**	**4 psi/s [0.028 MPa/s]**	**5 psi/s [0.034 MPa/s]**
Compared to	**1 psi/s [0.007 MPa/s]**		0.24452	0.55355	0.39907	0.19545
	2 psi/s [0.014 MPa/s]	0.24452		0.34917	0.54983	0.90375
	3 psi/s [0.021 MPa/s]	0.55355	0.34917		0.72462	0.34797
	4 psi/s [0.028 MPa/s]	0.39907	0.54983	0.72462		0.57328
	5 psi/s [0.034 MPa/s]	0.19545	0.90375	0.34797	0.57328	
	All results	0.25664	0.4377	0.69791	0.9439	0.42455

4.5. Effects of Creep

Initially, three specimens were used to assess the ultimate strength of the each batch and five specimens were to be tested at each stress level. However, due to the occurrence of premature failures in certain batches before reaching the specified stress, certain stress levels did not produce the expected amount of data. The ultimate strength of prematurely failed specimens were taken into account to reassess the average strength of the batch, and the stress levels were adjusted accordingly. The results of specimens that did not fall within the new stress levels were also included in the analysis.

All results are compiled in Figure 4-16. Since strain could not be measured in the present version of the pressure tension test, the time to failure was plotted with respect to the applied stress. The results for the separate batches are shown in Figure 4-17, Figure 4-18, and Figure 4-19, for 0.40 W/C, 0.50 W/C, and 0.60 W/C, respectively. As per the pre-imposed time limit for testing, the testing of 1 specimen of 0.50 W/C and of 1 specimen of 0.60 W/C were stopped as their loading time exceeded 40 minutes. Moreover, certain specimens were observed to experience a leak after a period of constant stress application. It is likely that this was due to the O-Rings trying to reposition themselves. Some O-Rings were twisted when fitted onto the specimen during the test preparation, and when they tried to reposition and untwist themselves after an extended period of load, they created an opening for a leak to occur. Testing of the specimens that experienced an excessive leak was immediately stopped. Although the software would attempt to compensate for the leak, it would generate a large fluctuation in the applied pressure so that the stress would no longer be constant. The results of all specimens that did not fail, either due to the time limit or the leak, are displayed as black data points in the graphs and labeled as 'Stopped'.

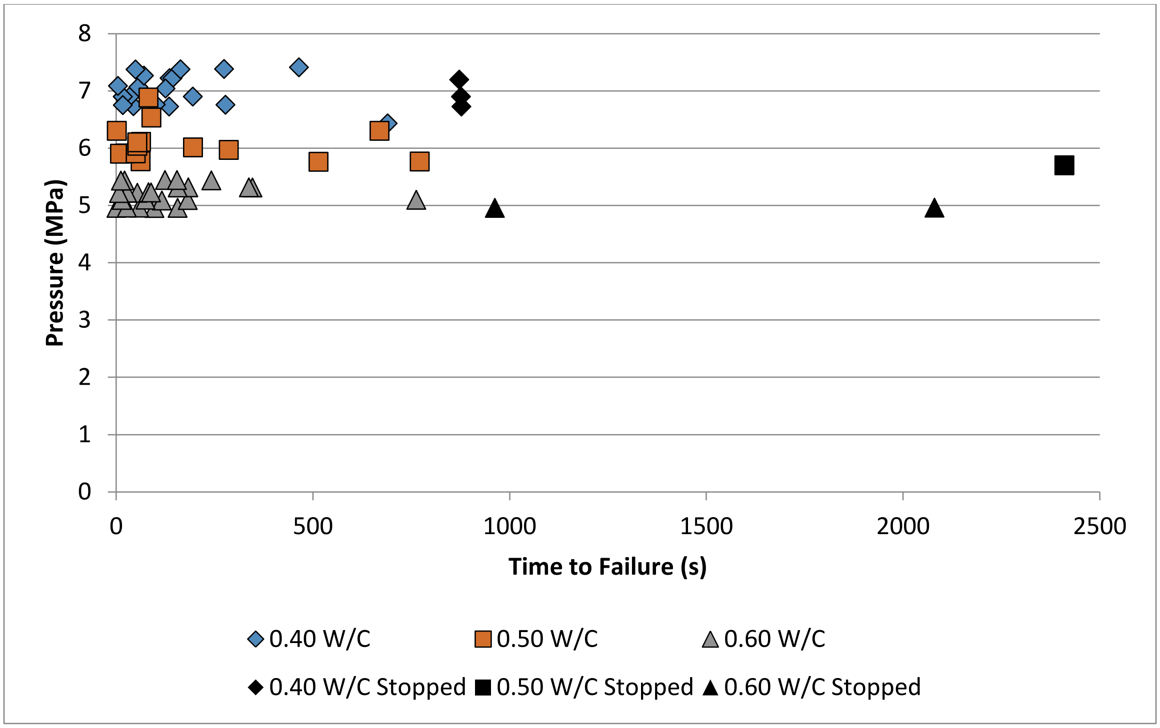


Figure 4-16 - Effects of Creep – Comparison of all Results

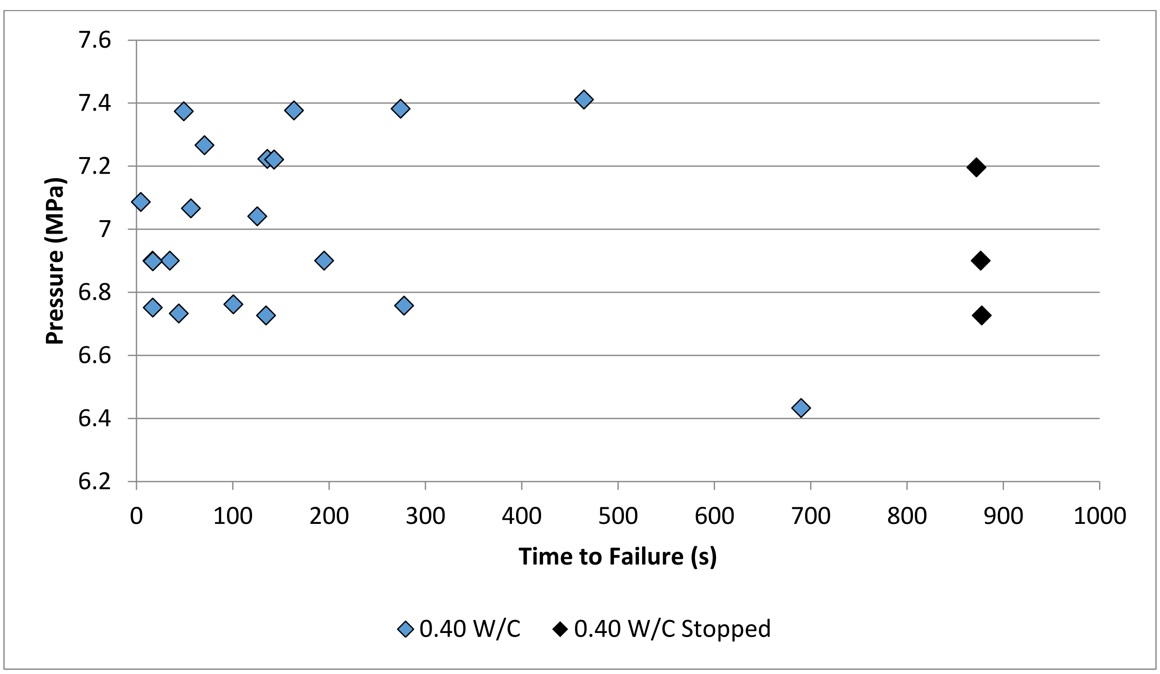


Figure 4-17 - Effects of Creep – Results of 0.40 W/C

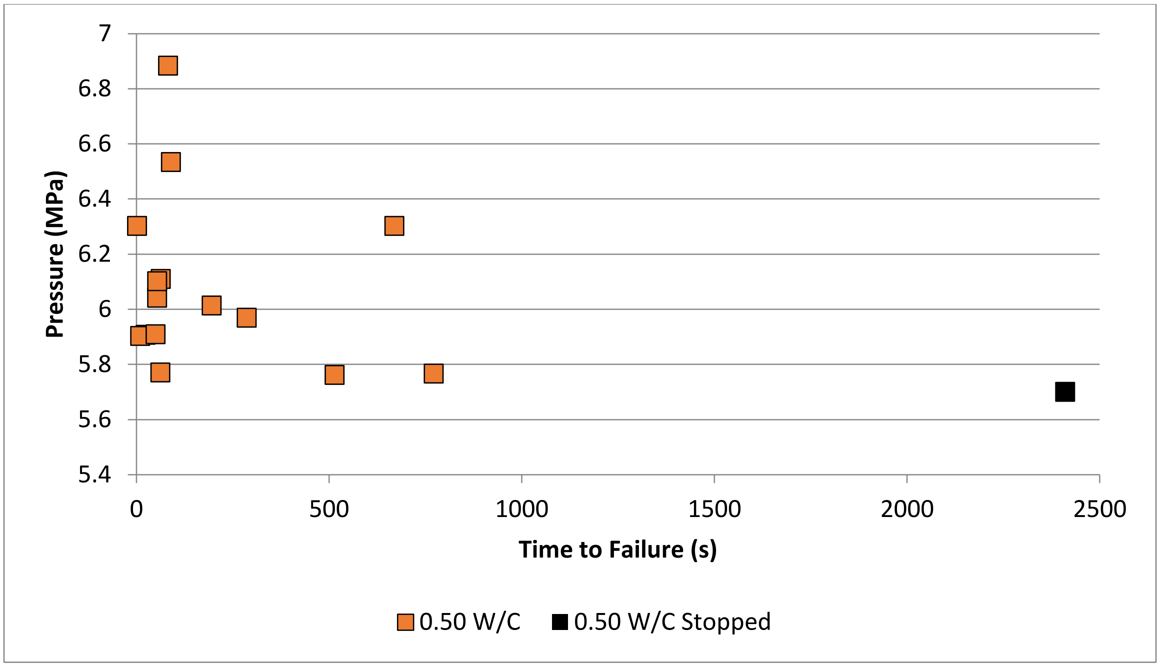


Figure 4-18 - Effects of Creep – Results of 0.50 W/C

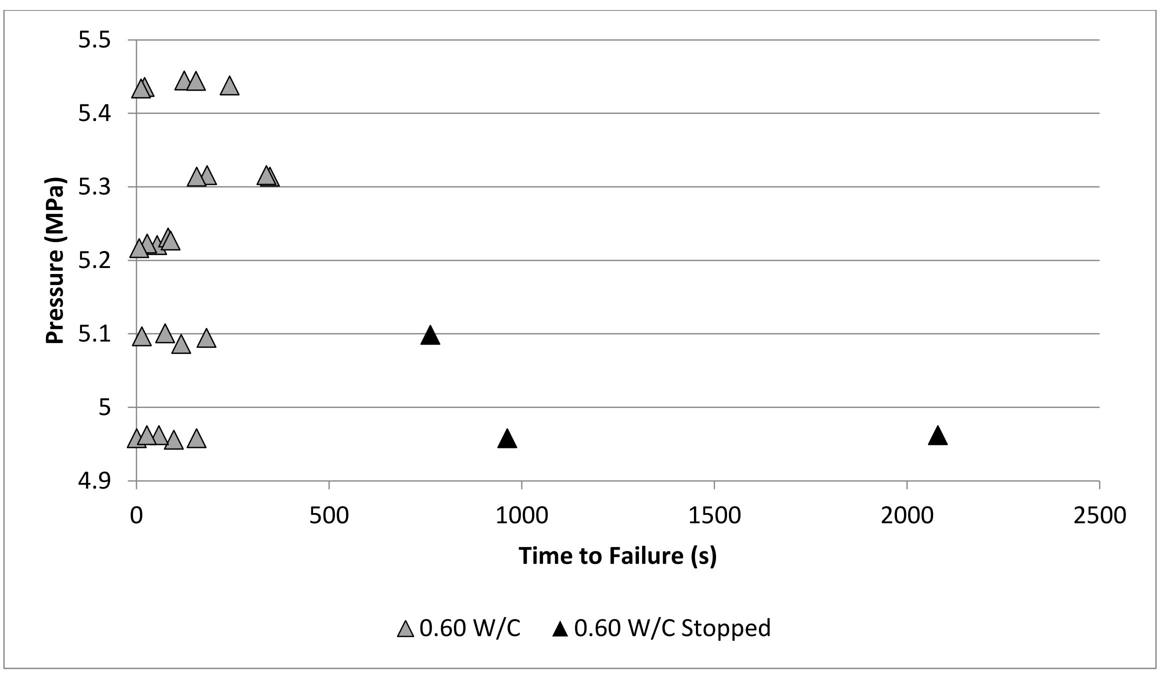


Figure 4-19 - Effects of Creep – Results of 0.60 W/C

Since the specimens were loaded at a stress level between 86% and 94%, the results were expected to fall within the failure envelope, as explained in Chapter 2. However, because strain was not measured in this research, it is difficult to justify whether the results depicted the failure limit. Assuming that the rate of strain increase would be constant for all specimens for the same batch, the time to failure would be expected to increase with a decrease in stress-to-strength ratio. However, the data points did not really show this. Also, the large variation in time to failure between specimens at the same constant stress level showed that there was variability in the specimens. More data points would be required for the tested range of stress levels.

Nonetheless, this series of test proved that the pressure tension test is capable of maintaining a constant applied stress for an extended period of time, allowing for the possibility of performing tension creep testing for concrete. This proof of concept demonstrates tension creep testing can indeed be performed with a very simple apparatus, free of mechanical irregularities. It is expected that in the near future, tension creep testing for concrete can be standardized with this new test method.

4.6. Failure Paths and Locations

All specimens tested in the pressure tension test failed in a plane relatively perpendicular to the axis. Some failure planes were more clean (Figure 4-20), cutting through the coarse aggregate particles, and others more irregular (Figure 4-21) as they passed around the coarse aggregate. This is because the pressure tension test exploits the path of least resistance of the specimen. The cleaner and smoother planes occurred primarily with specimens of higher strength, such as 0.40 W/C. The failure path tend to cut across the aggregates. Hence the matrix was, in this case, stronger than the aggregate. For specimens of lower strength, 0.50 W/C and 0.60 W/C, the failure path tended to lie at the interfacial-transition zone. Therefore, the resulting failure plane tended to be highly irregular and, sometimes, at a slight angle.

All specimens failed within the tested region. The location of fracture planes varied largely from specimen to specimen with 95.3% of the specimens failing within the middle 150 mm of the specimen's length. Another 2.3% failed at 25 mm (Figure 4-22) from the bottom, with respect to their upright position during curing, and 2.3% failed at 25 mm from the top (Figure 4-23), near the location of the O-Rings. The low rate of failure near the O-Ring implies that failure planes are not largely affected by the boundaries.

A few specimens were broken into many pieces (Figure 4-23 and Figure 4-24), potentially due to impact with the steel frame during the explosion when failure occurs. The limited amount of foam padding that was fit into the steel frame was not sufficient to stop the impact within the short amount of distance.

Figure 4-20 - Specimen 1-7
(a) Reconstructed (b) Separated

Figure 4-21 - Specimen 3-2
(a) Reconstructed (b) Separated

Figure 4-22 - Specimen 4-4-4
(a) Reconstructed (b) Separated

Figure 4-23 - Specimen 9-2-5
(a) Reconstructed (b) Separated

Figure 4-24 - Specimen 8-3-3
(a) Reconstructed (b) Separated

5. STANDARD TEST METHOD FOR PRESSURE TENSILE STRENGTH OF CYLINDRICAL CONCRETE SPECIMENS

5.1. Scope

5.1.1 This test method covers the determination of the pressure tensile strength of cylindrical concrete specimens, such as molded cylinders and drilled cores.

5.1.2 The values stated in either SI units or in-pound units are to be regarded separately as standard. The values stated in each system may not be exact equivalents; therefore, each system shall be used independently of the other. Combining values from the two systems may result in non-conformance with the standard.

5.1.3 *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.* (**Warning** – Means should be provided to contain concrete fragments during sudden rupture of specimens.)

5.2. Referenced Documents

5.2.1 ASTM Standards:

C31/C31M Practice for Making and Curing Concrete Test Specimens in the Field
C39/C39M Test Method for Compressive Strength of Cylindrical Concrete Specimens
C42/C42M Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C192/C192M Practice for Making and Curing Concrete Test Specimens in the Laboratory

5.3. Summary of Test Method

5.3.1 This test method consists of applying axisymmetrical compressive pressure along the length of a cylindrical concrete specimen at a rate that is within a prescribed range until failure occurs. This loading induces tensile stresses within the entire body of the specimen. Tensile failures occurs, rather than compressive failure, because the specimen is in a state of triaxial compression, in which the axial stress is reduced. Both the failure mechanism and the resulting failure mode are identical to an applied axial tension.

5.3.2 The maximum load sustained by the specimen is taken to be the pressure tensile strength.

5.4. Significance and Use

5.4.1 This test method is used to determine the pressure tensile strength of specimens prepared and cured in accordance with Practices C31/C31M and C192/C192M, and Test Method C42/C42M. The strength determined will vary where there are differences in specimen size, preparation, moisture condition, or curing.

5.4.2 The results of this test method may be used to determine compliance with specifications or as a basis for proportioning, mixing and placement operations.

5.4.3 Pressure tensile strength is generally greater than splitting tensile strength.

5.5. Apparatus

5.5.1 *Testing Machine* – The testing machine shall be of a type capable of providing the rates of stress prescribed in 5.7.3 and equipped with a source of gas pressure having sufficient capacity. The machine must consist at the minimum of a testing chamber fixed onto a protective frame, an adjustable valve, a source of gas pressure, such as a pressurized gas tank, and a device capable of recording the chamber pressure.

5.5.2 *Testing Chamber* – The testing chamber must be of a hollow cylindrical form with open ends. The internal diameter of the testing chamber must be 107 mm, which allows a concrete cylinder conforming to the Practices C31/C31M and C192/C192M and Test Method C42/C42M to be placed inside. It must be capable of isolating 150 mm to 160 mm of the specimen body from the ambient pressure by means of O-rings. The testing chamber must have an inlet connected to the source of gas pressure, and optionally an gas outlet connected to a safety valve for safety purposes. A diagram of an apparatus that accomplishes this purpose is shown in Figure 5-1. The chamber must be made of material capable of sustaining pressures up to 50 MPa without deformation. The testing chamber must be bolted or welded onto a protective frame.

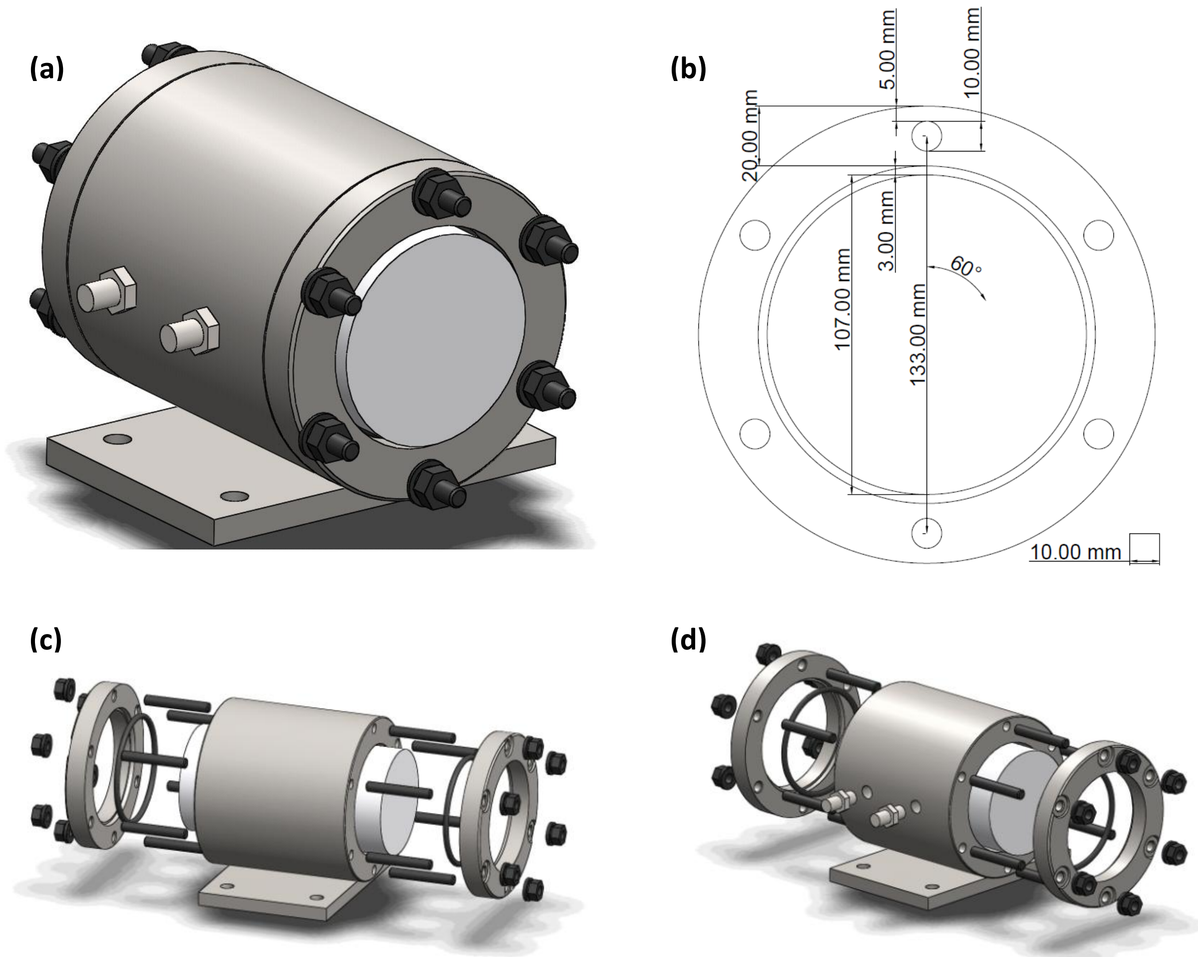


Figure 5-1 - Schedmatic of a Suitable Apparatus for Pressure Tension Test
(a) Unexploded rear view with gas inlet and outlet (b) Side view with dimensions
(c) Exploded front view (c) Exploded rear view with gas inlet and outlet

5.5.3 *Protective Frame* – The protective frame must be able to completely enclose the testing chamber and protect the operator of any flying debris during testing. It must be made of material capable of resisting large impact without large deformations.

5.5.4 *O-Rings* – New or undamaged circular O-Rings with an interior diameter of 100 mm and a thickness of 8 mm must be used. The thickness of the O-Rings must ensure a tight fit and prevent leaks. The O-Ring must be fitted onto the specimen without twisting.

NOTE 1 – Due to the explosive nature of the test, proper training by qualified personnel is highly recommended prior to the use of machine.

5.6. Test Specimens

5.6.1 The test specimens shall conform to the size, molding and curing requirements set forth in either Practice C31/C31M (field specimens) or Practice C192/C192M (laboratory specimens). Drilled cores shall conform to the size and moisture-conditioning requirements set forth in Test Method C42/C42M.

5.6.2 Moist-cured specimens shall be kept moist by any convenient method during the period between removal from moist storage and testing, and shall be tested in a moist condition as soon as practicable.

5.7. Procedure

5.7.1 *Wrapping* – Wrap the ends of the test specimen, at the approximate areas where O-rings will be placed, with PVC tape or equivalent. Excess surface moisture should be removed before taping.

5.7.2 *Placing the Specimen* – Place the specimen so as to ensure both O-rings cover areas of smooth surface or with the least amount of defects or holes.

5.7.3 *Rate of stress* – Apply the pressure continuously and without shock at a constant rate within the range of 0.4 to 2.0 MPa/min [60 to 300 psi/min] until failure of the specimen.

5.8. Report

5.8.1 Report the following information:

5.8.1.1 Specimen identification,

5.8.1.2 Specimen diameter and length, mm [in],

5.8.1.3 Pressure tensile strength, equivalent to the peak pressure obtained, MPa [psi],

5.8.1.4 Estimated proportion of coarse aggregate fractured during test, 5.8.1.5 Age of specimen,

5.8.1.6 Curing history

5.8.1.7 Defects in specimen

5.8.1.8 Path of fracture,

5.8.1.9 Location of fracture and

5.8.1.10 Type of specimen.

5.9. Keywords

5.9.1 cylindrical concrete specimens; pressure tension; tensile strength

6. CONCLUSION

The pressure tension test is a simple and effective method for determining the tensile strength of concrete. This study confirms the method is highly reliable in determining the tensile strength of concrete, and involves indirectly subjecting the concrete specimen to an equivalent condition to applied axial tension. Unlike other direct and indirect tension tests, the pressure tension test applies uniform stress throughout the entire tested portion of the concrete specimen, free of mechanical irregularities and secondary stress concentrations. In contrast to previous iterations of the apparatus, the device used in this research applied compressed air as loading fluid, because an air compressor was readily available allowing quick refills when the loading fluid became depleted. The results were not expected to be different from that of nitrogen gas, due to the similarity in properties. Consistent with the tensile strength measured using other methods, pressure tensile strength of undamaged concrete was found to lie between 10-16% of the compressive strength. The coefficient of variation (11-16%) of the new test method is comparable with that of the most common indirect tension test, the splitting tension test.

Variations in moisture conditions of the concrete specimens had an impact on the pressure tension test results. Similar to flexural strength, the pressure tensile strength experienced a sudden drop at all tested W/C once drying began. Pressure tensile strength decreased as much as two-thirds of the original strength for 0.40 W/C, and up to half the strength for 0.50 W/C and 0.60 W/C. Once drying began, the measured pressure tensile strength of dried specimens was relatively constant, over 8 to 9 days of drying. The immediate decrease in strength following the drying process is likely influenced by the drying of surficial moisture. However, the exact mechanism behind the weakening still requires further research. The influence of moisture conditions on the pressure tension test is a major disadvantage of the pressure tensile test compared to the splitting tension test.

The moisture content of test specimens poses a challenge to quantify, due to the explosive nature of the test method. This research attempted to measure the moisture content with a recovered portion of the original specimen. The measured moisture content varied up to 17.3% for specimens subjected to the same period of drying. This highlights the role of microstructural variations in the rate of drying for concrete. Moreover, moisture loss was detected in the pressure tension test. It was suspected that moisture was driven out of the specimen by the applied fluid pressure through the porous medium. The effect was more pronounced for stronger concrete, which is subjected to a higher fluid pressure. Consequently, the technique used in this study to measure concrete moisture content was not sufficiently accurate to establish a correlation between moisture content and pressure tensile strength. A more rigorous or a different testing strategy is required to accurately measure the moisture content for specimens undergoing the pressure tension test.

The stress rate determines whether a tested subject fails at the true pressure tensile strength of the material, or is subject to creep. In this research, a stress rate range of 1 psi/s [0.007 MPa/s] to 5 psi/s [0.034 MPa/s] was tested to provide consistent pressure tensile results. Although apparent variations were observed for certain results, in-depth statistical analysis based on ANOVA and the Student's t-test suggested that all results belonged to the same population for all three tested W/C. Repeatability can therefore be achieved within this range of stress rates. However, further research is required to confirm this conclusion. Stress rates beyond the studied range must also be investigated to determine the complete range for which pressure tensile strength results are consistent.

Moreover, this research demonstrated that the pressure tension test is capable of maintaining a constant applied stress condition, thus confirming the test’s utility for measuring tensile creep. Tests were performed at stress levels within the failure envelope to evaluate the time to failure. Unfortunately, certain data points were lost due to premature failure of some concrete specimens, which required reevaluation of the ultimate strength of certain batches. The time to failure varied drastically at every applied stress level, from a few seconds up to tens of minutes. A few specimens did not fail within the preset time limit of 40 minutes which required the test to be stopped. Since strain was not measured, it cannot be ruled out that similar strain limits exist for each of the concrete specimens. Future studies must include a strain gage or other means to detect changes in length. Nonetheless, the experiment did act as a proof of concept study, which implies that this test method can be standardized in the future to include concrete creep assessment in tension.

Following a prolonged period of constant stress application, fluid leak was observed for certain test subjects. The loosening of twists derived from the improper fitting during specimen preparation is exacerbated by the elastic nature of O-rings. Careful preparation is therefore paramount to ensure reliable pressure tension creep results.

In the pressure tension test, specimen failure occurred at the surface of least resistance. This weakest plane was located predominantly across the interfacial-transition zone for specimens with lower strength, at 0.50 W/C and 0.60 W/C, and tended to be relatively irregular. The failure path cut across aggregates for specimens of higher strength, at 0.40 W/C, because the matrix was stronger. A smooth and clean failure surface was often observed. The majority of the specimens failed in the middle 150 mm of the specimen, proving that the failure planes are not largely affected by the boundaries. The remains of the specimen were often broken into multiple pieces due to the impact with the steel frame during the explosion when failure occurs.

Based on all available literature and the findings in this research, a draft standard test method with an emphasis on health and safety concerns was developed. The explosive nature of the test necessitates careful training prior to the use of a pressure tension apparatus. Moreover, the draft standard was prepared with a sense of universality in terms of design, which was meant to permit future improvements to the configuration of the pressure tension apparatus. The standard was prepared for both cast cylinders and drilled cores. Preparation methods and materials specified were consistent with those of all previous and present iterations of the device.

The development of the pressure tension test is still at an early stage. Yet, it has already demonstrated its capability in evaluating durability in concrete. The preparation process is marginally more involved than common tension tests, such as the splitting tension test, making it a potential candidate for implementation as an industry standard. However, while this analysis shows that the new test method is deemed highly practical, the pressure tension test is also theoretically more reliable because it subjects a greater portion of the specimen to the test conditions. The observations and conclusions of this present research are intended to serve as a preliminary knowledge base for future research and development of the test method. The pressure tension test has tremendous potential to become a specification criterion alongside compressive strength in future concrete designs.

REFERENCES

Abrams, D. A. (1918). *Design of Concrete Mixtures*. Structural Materials Research Laboratory. Chicago: Lewis Institute.

Al-Kubaisy, M. A., & Young, A. G. (1975). *Failure of Concrete under Sustained Tension.* Magazine of Concrete Research, 27(92), 171-178.

American Concrete Institute (ACI) Committee 224--Cracking. (1997). *Cracking of concrete members in direct tension*. Farmington Hills, Mich.: American Concrete Institute.

American Society for Testing and Materials (ASTM). (2011). *Standard Practice for Making and Curing Concrete test Specimens in the Field*, C31/C31M-10 (pp. 6-11). West Conshohocken, Pennsylvania: ASTM International.

American Society for Testing and Materials (ASTM). (2011). *Standard Specification for Concrete Aggregates*, C33/C33M-11a (pp. 11-22). West Conshohocken, Pennsylvania: ASTM International.

American Society for Testing and Materials (ASTM). (2011). *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*, C39/C39M-11 (pp. 23-29). West Conshohocken, Pennsylvania: ASTM International.

American Society for Testing and Materials (ASTM). (2011). Sta*ndard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)*, C78/C78M-10 (pp. 43-46). West Conshohocken, Pennsylvania: ASTM International.

American Society for Testing and Materials (ASTM). (2011). *Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate*, C127-12. West Conshohocken, Pennsylvania: ASTM International.

American Society for Testing and Materials (ASTM). (2011). *Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Fine Aggregate*, C128-12. West Conshohocken, Pennsylvania: ASTM International.

American Society for Testing and Materials (ASTM). (2011). *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates*, C136-06. West Conshohocken, Pennsylvania: ASTM International.

American Society for Testing and Materials (ASTM). (2011). *Standard Test Method for Slump of Hydraulic-Cement Concrete*, C143/C143M-12. West Conshohocken, Pennsylvania: ASTM International.

American Society for Testing and Materials (ASTM). (2011). *Standard Specification for Portland Cement,* C150/C150M-12. West Conshohocken, Pennsylvania: ASTM International.

American Society for Testing and Materials (ASTM). (2011). *Standard Practice for Sampling Freshly Mixed Concrete*, C172/C172M-10. West Conshohocken, Pennsylvania: ASTM International.

American Society for Testing and Materials (ASTM). (2011). *Standard Practice for Making Curing Concrete Test Specimens in the Laboratory*, C192-C192M-07 (pp. 141-148). West Conshohocken, Pennsylvania: ASTM International.

American Society for Testing and Materials (ASTM). (2011). *Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Center-Point Loading),* C293/C293M-10 (pp. 192-194). West Conshohocken, Pennsylvania: ASTM International.

American Society for Testing and Materials (ASTM). (2011). *Standard Specification for Molds for Forming Concrete Test Cylinders Vertically*, C470/C470M-09. West Conshohocken, Pennsylvania: ASTM International.

American Society for Testing and Materials (ASTM). (2011). *Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens*, C496/C496M-11 (pp. 301-305). West Conshohocken, Pennsylvania: ASTM International.

American Society for Testing and Materials (ASTM). (2011). *Standard Test Method for Direct Tensile Strength of Intact of Intact Rock Core Specimens,* D2936-08 (pp. 323-325). West Conshohocken, Pennsylvania: ASTM International.

Arellano, D., & Thompson, M. R. (1998*). Stabilized Base Properties (Strength, Modulus, Fatigue) for Mechanistic-Based Airport Pavement Design* (D. o. C. Engineering, Trans.). Urbana, Illinois: University of Illinois at Urbana-Champaign.

Arιoglu, N., Girgin, Z. C., & Arιoglu, E. (2006). *Evaluation of Ratio between Splitting Tensile Strength and Compressive Strength for Concretes up to 120 MPa and its Application in Strength Criterion*. ACI Materials Journal, 103(M03), 18-24.

Atrushi, D. S. (2003). *Tensile and Compressive Creep of Early Age Concrete: Testing and Modelling*. (Doctoral Thesis), The Norwegian University of Science and Technology, Trondheim, Norway.

Bamforth, P. B. (1987). *The relationship between permeability coefficients for concrete obtained using liquid and gas*. Magazine of Concrete Research, 39(138), 3-11.

Bolzan, P. E., & Huber, G. (1993). *Direct Tension Test Experiments*. Austin, Texas: The University of Texas at Austin.

Boyd, A., & Mindess, S. (2001). *The Effect of Sulfate Attack on the Tensile to Compressive Strength Ratio of Concrete*. Paper presented at the Third International Conference on Concrete under Severe Conditions, Vancouver.

Bremner, T. W., Boyd, A. J., Holm, T. A., & Boyd, S. R. (1995). *Tensile Testing to Evaluate the Effect of Alkali-Aggregate Reaction in Concrete.* Paper presented at the International Workshop on Alkali-Aggregate Reactions in Concrete, Dartmouth, Canada.

Bridgman, P. W. (1931). *The physics of high pressure* ([1st ed.). New York,: Dover

Brooks, J. J., & Neville, A. M. (1977). *A comparison of creep, elasticity and strength of concrete in tension and in compression*. Magazine of Concrete Research, 29(100), 131-141.

Bungey, J. H., SMillard, S. G., & Grantham, M. G. (2006). *Testing of Concrete in Structures* (4th Ed. ed.). Oxon: Taylor & Francis.

Canadian Standards Association (CSA). (2003). *Cementitious Materials for Use in Concrete*, CAN/CSA-A3001-03. Mississauga, Ontario.

Canadian Standards Association (CSA). (2006). *Concrete Materials and Methods of Concrete Construction*, CAN/CSA-A23.1-04. Mississauga, Ontario.

Canadian Standards Association (CSA). (2009). *Splitting Tensile Strength of Cylindrical Concrete Specimens*, CAN/CSA A23.2-13C. Mississauga, Ontario.

Cantillo, V., & Guzmán, A. (2013). *Fluid Pressured Test to Measure Tensile Strength of Concrete.* Journal of Materials in Civil Engineering.

Carino, N. J., & Lew, H. S. (1982). *Re-Examination of the Relation Between Splitting Tensile and Compressive Strength of Normal Weight Concrete*. *ACI Journal, 88*(2), 214-219.

Carrasquillo, P. M., & Carrasquillo, R. L. (1987). *Improved Concrete Quality Control Procedures Using Third Point Loading:* University of Texas at Austin Center for Transportation Research.

Chen, X., Huang, W., & Zhou, J. (2012). *Effect of Moisture Content on Compressive and Split Tensile Strength of Concrete*. Indian Journal of Engineering & Materials Sciences, 19, 427-435.

Clayton, N. (1978). *Fluid-pressure testing of concrete cylinders*. Magazine of Concrete Research, 30(102), 26-30.

Clayton, N., & Grimer, F. J. (1979). *The Diphase Concept, with Particular Reference to Concrete Developments in Concrete Technology* (pp. 283-318). Watford, UK: Applied Science Publishers.

Do, Q., Orr, R., Wojcik, C., Cumming, S., Krahn, T., & Kyriakelis. (2001). *Tensile Strength of Concrete CIVE 321 Laboratory Materials Course*: University of British Columbia.

Domone, P. L. (1974). *Uniaxial Tensile Creep and Failure of Concrete*. Magazine of Concrete Research, 26(88), 144-152.

Fairbairn, E. M. R., & Ulm, F.-J. (2002). *A Tribute to Fernando L. L. B. Carneiro (1913 - 2001) Engineer and Scientist who invented the Brazilian Test.* Materials and Structures/Matériaux et Constructions*, 35*, 195-196.

Fujikake, K., Mindess, S., Uno, T., & Xu, H. (2010). *The Nitrogen Gas Tension Test. Part 2: Failure Mechanism.* Materials and Structures, 44(4), 865-877.

Glanville, W. H., & Thomas, F. G. (1939). *Studies in reinforced concrete - IV. Further investigations on the creep or flow of concrete under load*. Building Research Technical Paper, 21.

Gonnerman, H., & Shuman, E. (1928). *Compression, Flexural and Tension Tests of Plain Concrete*. ASTM Proceedings, 28, 527-564.

Graybeal, B. A., & Baby, F. (2013). *Development of Direct Tension Test Method for Ultra-HighPerformance Fiber-Reinforced Concrete*. ACI Materials Journal, 110 (M17), 177-186.

Grimer, F. J., & Hewitt, R. E. (1969). *The form of the stress-strain curve of concrete interpreted with a diphase concept of material behaviour.* Paper presented at the International Conference in Civil Engineering Materials.

Hannant, D. J., Buckley, K. J., & Croft, J. (1973). *The effect of aggregate size on the use of the cylinder splitting test as a measure of tensile strength*. Materials and Structures/Matériaux et Constructions*, 6*, 15-21.

Haroun, W. A. (1968). *Uniaxial Tensile Creep and Failure of Concrete*. (Doctor of Philosophy in Civil Engineering), University of London, London, England.

Hartell, J., Boyd, A. J., & Ferraro, C. C. (2011). *Sulfate Attack on Concrete: Effect of Partial Immersion*. Journal of Materials in Civil Engineering, 23(5), 572-579.

Hudson, W. R., & Kennedy, T. W. (1968). *An Indirect Tensile Test for Stabilized Materials* (C. f. H. Research, Trans.) Evaluation of Tensile Properties of Subbases for Use in New Rigid Pavement Design: The University of Texas at Austin.

Illston, J. M. (1965). *The creep of concrete under uniaxial tension*. Magazine of Concrete Research, 17(51), 77-84.

Johnston, C. D., & Sidwell, E. H. (1969). *Influence of Drying on Strength of Concrete Specimens*. American Concrete institute Journal Proceedings, 66, 748-755.

Kadlecek, V., Modry, S., & Kadlecek, V. J. (2002). *Size Effect of Test Specimens on Tensile Splitting Strength of Concrete: General Relation*. Materials and Structures/Matériaux et Constructions, 35, 28-34.

Komar, A. J. K., & Boyd, A. J. (2014, July 21 to 23, 2014). *Pressure-Tension Testing in the Evaluation of Freeze-Thaw Deterioration*. Paper presented at the 10th fib International PhD Symposium in Civil Engineering, Université Laval, Québec, Canada.

Komar, A. J. K., Hartell, J. A., & Boyd, A. J. (2014). *Pressure Tension Test: Reliability for Assessing Concrete Deterioration*.

Kosmatka, S. H., Kerkhoff, B., & Panarese, W. C. (2002a). Chapter 4 - Mixing Water for Concrete. *Design and Control of Concrete Mixtures* (14 ed.). Skokie, Illinois: Portland Cement Association.

Kosmatka, S. H., Kerkhoff, B., & Panarese, W. C. (2002b). Chapter 9 - Designing and Proportioning Normal Concrete Mixtures. *Design and Control of Concrete Mixtures* (14 ed.). Skokie, Illinois: Portland Cement Association.

Langan, D., & Garas, F. K. (1969). *The failure of concrete under the combined action of high shearing forces and biaxial restraint*. Paper presented at the Internat. Conf. Structure, Solid Mechanics and Engrg. Design in Civil Engrg., Southampton.

Li, G. (2004). *The Effect of Moisture Content on the Tensile Strength Properties of Concrete.* (Master of Engineering), University of Florida, Gainesville, Fla.

Lin, Z., & Wood, L. (2003). *Concrete uniaxial tensile strength and cylinder splitting test.* Journal of Structural Engineering (ASCE), 129(5), 692-698.

Malárics, V. M., H.S. . (2010). *Evaluation of the splitting tension test for concrete from a fracture mechanical point of view*. Paper presented at the Fracture Mechanics of Concrete and Concrete Structures - Assessment, Durability, Monitoring and Retrofitting of Concrete Structures, Seoul, Republic of Korea.

Mamillan, M. (1959). *A study of the creep of concrete*. RILEM Bulletin, 3, 15-31.

Neville, A. M., Dilger, W. H., & Brooks, J. J. (1983). *Creep of plain and structural concrete*. London ; New York: Construction Press.

Neville, A. M. (1996). *Properties of Concrete* (4 ed.). New York: John Wiley & Sons, Inc.

Nielsen, K. E. C. (1954). *Effect of Various Factors on Flexural Strength of concrete Test Beams*. Magazine of Concrete Research, 15, 105-114.

Ozyildirim, C., & Carino, N. J. (2006). *Concrete Strength Testing.* In J. F. Lamond & J. H. Pielert (Eds.), Significance of tests and properties of concrete and concrete-making materials. Philadelphia, PA: ASTM.

Pann, K. S., Yen, T., Tang, C. W., & Lin, T. D. (2003). *A New Strength Model Based on Water/Cement Ratio and Capillary Porosity*. ACI Materials Journal, 100(4), 311-318.

Richart, F. E., Brandtzaeg, A., & Brown, R. L. (1928). *A study of the failure of concrete under combined compressive stresses*. (Vol. Bulletin No. 185, pp. 104). Urbana: University of Illinois, Engineering Experiment Station.

Rusch, H. (1960). *Researches Toward a General Flexural Theory for Structural Concrete*. ACI Journal Proceedings, 57(7).

Swaddiwudhipong, S., Lu, H.-R., & Wee, T.-H. (2003). *Direct tension test and tensile strain capacity of concrete at early age.* Cement and Concrete Research, 33(12), 2077-2084.

Uno, T., Fujikake, K., Mindess, S., & Xu, H. (2010). *The nitrogen gas tension test of concrete. Part 1: effect of boundary conditions and axial strain response*. Materials and Structures, 44(4), 857-864.

US Bureau of Reclamation. (1953). *A ten-year study of creep properties of concrete Concrete Laboratory Report* (pp. 14). Denver, Colorado.

van Mier, J. G. M., & van Vliet, M. R. A. (2002). *Uniaxial tension test for the determination of fracture parameters of concrete: state of the art*. Engineering Fracture Mechanics, 69, 235-247.

Appendix A – Fine Aggregate Analysis

SIEVE ANALYSIS

sieve size (mm)	Sieve Mass (g)	Sieve and Retained Mass (g)	Retained Mass (g)	% Retained	Cumulative % Retained	% Passing
5	458.36	499.89	41.53	8.31	8.31	91.69
2.5	407.33	450.95	43.62	8.72	17.03	82.97
1.25	383.83	442.5	58.67	11.73	28.77	71.23
0.63	336.99	441.31	104.32	20.87	49.63	50.37
0.315	307.17	445.46	138.29	27.66	77.29	22.71
0.16	297.17	375.35	78.18	15.64	92.93	7.07
0.08	288.22	315.3	27.08	5.42	98.35	1.65
Pan	279.27	287.54	8.27	1.65	100.00	0.00
Total	2758.34	3258.3	499.96			

Mass sample before sieving :	500.17 g
Total mass of sample after sieving	499.96 g
Difference:	0.21 g
Fineness modulus:	2.74

SPECIFIC GRAVITY AND ABSORPTION

Determine mass of sample

Mass of pycnometer and sample	1148.43 g
Mass of pycnometer	648.85 g
Mass of sample [D]	499.58 g
Mass of pycnometer filled with sample and water [C]	1748.23 g
Mass of pycnometer filled with water [B]	1451.09 g

Determine mass of oven-dry sample

Mass of pan and oven-dry sample	2606.40 g
Mass of pan	2112.70 g
Mass of oven dry sample [A]	493.70 g

Bulk SG dry	A / (B + D - C)	2.439
Bulk SG SSD	D / (B + D - C)	2.468
Apparent SG	A / (B + A - C)	2.512
Absorption	[(D - A) / A] x 100%	1.19 %

Appendix B – Coarse Aggregate Analysis

SIEVE ANALYSIS

sieve size (mm)	Sieve Mass (g)	Sieve and Retained Mass (g)	Retained Mass (g)	% Retained	Cumulative % Retained	% Passing
27	6792.5	6795.5	3	0.10	0.10	99.90
20	6912.5	6914.5	2	0.07	0.17	99.83
14	6984	7252.5	268.5	8.98	9.15	90.85
10	7085.5	8238	1152.5	38.56	47.72	52.28
5	6785.5	8257	1471.5	49.24	96.95	3.05
2.5	6425	6465	40	1.34	98.29	1.71
Pan	5928	5979	51	1.71	100.00	0.00
Total	46913	49901.5	2988.5			

Mass sample before sieving :	3000 g
Total mass of sample after sieving:	2988.5 g
Difference:	11.5 g

SPECIFIC GRAVITY AND ABSORPTION

Determine mass of saturated sample in water	
Mass of basket and sample	3032.5 g
Mass of basket	878.3 g
Mass of saturated sample in water [C]	2154.2 g
Determine mass of saturated surface dry sample	
Mass of pan and sample	5512.5 g
Mass of pan	2113.1 g
Mass of SSD sample [B]	3399.4 g
Determine mass of oven-dry sample	
Mass of pan and sample	5494.0 g
Mass of pan	2114.5 g
Mass of oven dry sample [A]	3379.5 g

Bulk SG dry	A / (B - C)	2.714
Bulk SG SSD	B / (B - C)	2.730
Apparent SG	A / (A - C)	2.758
Absorption	[(B - A) / A] x 100%	0.59 %

DRY RODDED UNIT WEIGHT OF COARSE AGGREGATE

Volume of bucket [A]		$0.01416 m^3$
Determine mass of dry rodded aggregate		
Mass of bucket and dry rodded sample		27.6 kg
Mass of bucket		5.4075 kg
Mass of dry rodded aggregates [B]		22.1925 kg
Unit weight	B / A	$1567.44 kg/m^3$

Appendix C – Mixture Design

Reference Guide: Design and Control of Concrete Mixtures, Chapter 9 - Designing and Proportioning Normal Concrete Mixtures.

MATERIAL PROPERTIES

Cement		**Coarse Aggregate**	
Type	GU	Nominal maximum-size	14 mm
Relative density	3	Oven-dry relative density	2.714
Fine Aggregate		Absorption	0.59 %
Oven-dry relative density	2.43875	Oven-dry rodded bulk density	1567.44 kg/m3
Absorption	1.19%	Moisture content	0 %
Moisture content	0 %		
Fineness modulus	2.7	**Super Plasticizer**	5 ml/kg cement

MIXTURE PROPORTION CALCULATION

W/C	**0.40**	**0.50**	**0.60**
Target Volume	1.000 m^3	1.000 m^3	1.000 m^3
Water (Before adjusting for aggregate absorption)			
Mass (From Table 9-5)	213.46 kg/m^3	213.46 kg/m^3	213.46 kg/m^3
Mass (With superplasticizer, -15%)	181.44 kg/m^3	181.44 kg/m^3	181.44 kg/m^3
Volume	0.181 m^3	0.181 m^3	0.181 m^3
Cement			
Mass (Based on W/C)	453.61 kg/m^3	362.88 kg/m^3	302.40 kg/m^3
Volume	0.151 m^3	0.121 m^3	0.101 m^3
Coarse Aggregate			
Volume Proportion (From Table 9-4)	0.576	0.576	0.576
Mass	903.09 kg/m^3	903.09 kg/m^3	903.09 kg/m^3
Volume	0.333 m^3	0.333 m^3	0.333 m^3
Superplasticizer			
Mass	2.27 L/m^3	1.81 L/m^3	1.51 L/m^3
Volume	0.002 m^3	0.002 m^3	0.002 m^3
Fine Aggregate			
Mass (1 - sum of other materials)	0.332 m^3	0.363 m^3	0.383 m^3
Quantity	897.31 kg/m^3	980.18 kg/m^3	1035.43 kg/m^3
Water (adjusted for aggregate absorption)			
Mass	195.18 kg/m^3	196.62 kg/m^3	197.58 kg/m^3
Volume	0.195 m^3	0.197 m^3	0.198 m^3

FINAL MIXTURE PROPORTION

W/C	**0.40**	**0.50**	**0.60**
Water	195.18 kg/m3	196.62 kg/m3	197.58 kg/m3
Cement	453.61 kg/m3	362.88 kg/m3	302.40 kg/m3
Coarse	903.09 kg/m3	903.09 kg/m3	903.09 kg/m3
Fine	897.31 kg/m3	980.18 kg/m3	1035.43 kg/m3
Super Plasticizer	2.27 L/m3	1.81 L/m3	1.51 L/m3
TOTAL	**2451.45 kg/m3**	**2444.59 kg/m3**	**2440.01 kg/m3**

Appendix D – Pressure Tension Results

Series 1 – Variability in Test Method								
NOTE – Specimen named in the following method:					Batch # - Sequence of Testing			
Batch 1 (0.40 W/C)			Batch 2 (0.50 W/C)			Batch 3 (0.60 W/C)		
Specimen Name	Failure Stress (MPa)	Note	Specimen Name	Failure Stress (MPa)	Note	Specimen Name	Failure Stress (MPa)	Note
1-1	9.404		2-1	6.311		3-1	5.003	
1-2	9.581		2-2	6.431		3-2	4.557	
1-3	8.610		2-3	5.505		3-3	5.332	
1-4	9.448		2-4	5.364		3-4	4.812	
1-5	8.697		2-5	6.444		3-5	5.020	
1-6	8.522		2-6	5.352		3-6	5.561	
1-7	9.528		2-7	5.740		3-7	4.955	
1-8	10.108		2-8	6.473		3-8	4.214	
1-9	8.387		2-9	4.415		3-9	5.417	
1-10	8.868		2-10	5.096		3-10	5.400	
1-11	8.447		2-11	5.314		3-11	6.454	
1-12	9.219		2-12	5.067		3-12	4.421	
1-13	8.317		2-13	5.985		3-13	5.183	
1-14	8.214		2-14	4.817		3-14	4.032	
1-15	8.496		2-15	4.760		3-15	4.595	
1-16	8.721		2-16	5.801		3-16	4.647	
1-17	10.438		2-17	4.618		3-17	4.872	
1-18	8.710		2-18	6.013		3-18	4.691	
1-19	9.588		2-19	5.984		3-19	4.415	
1-20	10.265		2-20	6.115		3-20	5.576	
1-21	12.149		2-21	5.045		3-21	5.223	
1-22	14.440		2-22	5.986		3-22	3.815	
1-23	11.963		2-23	5.602		3-23	4.349	
1-24	8.813		2-24	4.967		3-24	5.524	
1-25	8.037		2-25	5.778		3-25	5.024	
						3-26	5.000	

Series 2 – Effects of Moisture Content

NOTE – Specimen named in the following method: Batch # - Days of Drying - Sequence of Testing

Batch 4 (0.40 W/C)					Batch 5 (0.50 W/C)				Batch 6 (0.60 W/C)			
Specimen Name	**Relative Moisture Content (%)**	**Normalized Relative Moisture Content (%)**	**Failure Stress (MPa)**	**Note**	**Specimen Name**	**Relative Moisture Content (%)**	**Failure Stress (MPa)**	**Note**	**Specimen Name**	**Relative Moisture Content (%)**	**Failure Stress (MPa)**	**Note**
4-0-1	104.275	104.275	7.696	Saturated	**5-0-1**	98.994	4.212	Saturated	**6-0-1**	98.749	4.523	Saturated
4-0-2	100.801	100.801	6.417	Saturated	**5-0-2**	99.289	4.206	Saturated	**6-0-2**	100.186	4.887	Saturated
4-0-3	97.101	97.101	6.833	Saturated	**5-0-3**	102.206	4.468	Saturated	**6-0-3**	99.011	5.368	Saturated
4-0-4	100.102	100.102	9.095	Saturated	**5-0-4**	98.324	4.325	Saturated	**6-0-4**	103.452	4.142	Saturated
4-0-5	97.721	97.721	9.101	Saturated	**5-0-5**	101.187	5.136	Saturated	**6-0-5**	98.602	4.513	Saturated
4-1-1	107.108	95.747	4.213		**5-1-1**	89.669	2.873		**6-1-1**	83.060	3.360	
4-1-2	108.333	96.972	4.182		**5-1-2**	89.985	3.210		**6-1-2**	87.856	2.791	
4-1-3	89.307	77.946	4.622		**5-1-3**	88.382	3.389		**6-1-3**	94.525	3.688	
4-1-4	94.336	82.975	4.202		**5-1-4**	101.924	2.986		**6-1-4**	93.450	3.610	
4-1-5	92.248	80.887	3.843		**5-1-5**	100.961	3.331		**6-1-5**	90.700	2.911	
4-2-1	97.116	85.755	3.084		**5-2-1**	95.494	3.009		**6-2-1**	93.987	2.398	
4-2-2	100.074	88.713	3.619		**5-2-2**	88.586	2.545		**6-2-2**	92.874	3.492	
4-2-3	101.711	90.350	3.648		**5-2-3**	96.316	3.087		**6-2-3**	89.438	2.784	
4-2-4	105.192	93.831	3.450		**5-2-4**	93.851	2.928		**6-2-4**	92.803	3.165	
4-2-5	100.138	88.777	3.060		**5-2-5**	96.830	2.702		**6-2-5**	92.149	2.776	
4-3-1	97.534	86.173	2.709		**5-3-1**	94.310	2.732		**6-3-1**	104.115	2.838	
4-3-2	94.555	83.194	3.182		**5-3-2**	87.604	3.071		**6-3-2**	87.187	2.908	
4-3-3	97.492	86.131	3.534		**5-3-3**	83.287	2.608		**6-3-3**	81.727	3.100	
4-3-4	97.132	85.771	3.466		**5-3-4**	79.282	3.712		**6-3-4**	85.863	3.363	
4-3-5	96.685	85.324	3.431		**5-3-5**	81.398	2.756		**6-3-5**	88.660	3.145	
4-4-1	74.279	62.918	2.731		**5-4-1**	87.729	2.452		**6-4-1**	87.521	2.566	
4-4-2	77.584	66.223	2.508		**5-4-2**	102.985	3.089		**6-4-2**	94.482	2.879	
4-4-3	107.407	96.046	2.996		**5-4-3**	93.286	2.776		**6-4-3**	95.766	3.319	
4-4-4	106.956	95.595	3.144		**5-4-4**	97.443	2.869		**6-4-4**	92.943	2.613	
4-4-5	100.550	89.189	3.279		**5-4-5**	97.482	3.346		**6-4-5**	96.409	3.155	
4-5-1	108.982	97.621	2.322		**5-5-1**	95.787	3.048		**6-5-1**	94.622	2.527	
4-5-2	107.117	95.756	3.529		**5-5-2**	90.983	2.919		**6-5-2**	88.220	3.213	
4-5-3	98.936	87.575	3.129		**5-5-3**	97.481	2.626		**6-5-3**	91.668	2.634	
4-5-4	111.361	100.000	3.667		**5-5-4**	92.965	3.058		**6-5-4**	95.477	3.260	
4-5-5	108.263	96.902	3.283		**5-5-5**	96.280	3.060		**6-5-5**	97.134	2.917	
4-6-1	107.714	96.353	2.797		**5-5-6**	98.682	2.172		**6-6-1**	93.929	3.295	
4-6-2	104.213	92.852	2.420		**5-6-1**	96.553	2.721		**6-6-2**	93.900	2.903	
4-6-3	101.605	90.244	2.429		**5-6-2**	95.198	3.117		**6-6-3**	97.618	3.335	
4-6-4	107.033	95.672	3.443		**5-6-3**	98.504	2.745		**6-6-4**	92.678	2.895	
4-6-5	107.007	95.646	2.834		**5-6-4**	93.154	3.365		**6-6-5**	87.657	3.358	
4-7-1	102.742	91.380	2.343		**5-6-5**	94.289	2.720		**6-7-1**	83.877	3.206	
4-7-2	104.321	92.960	2.867		**5-7-1**	95.272	2.720		**6-7-2**	92.198	2.878	
4-7-3	107.365	96.004	2.670		**5-7-2**	94.807	2.794		**6-7-3**	92.334	3.018	
4-7-4	101.338	89.977	3.014		**5-7-3**	92.527	2.830		**6-7-4**	94.264	2.940	
4-7-5	102.362	91.001	3.293		**5-7-4**	95.097	2.587		**6-7-5**	92.073	3.147	
4-8-1	87.559	76.198	3.021		**5-7-5**	95.076	3.078		**6-8-1**	78.361	2.992	
4-8-2	80.768	69.407	2.750		**5-8-1**	79.284	3.291		**6-8-2**	78.052	3.129	
4-8-3	83.127	71.766	2.675		**5-8-2**	83.062	3.111		**6-8-3**	75.762	3.139	
4-8-4	87.319	75.958	2.461		**5-8-3**	82.679	3.198		**6-8-4**	76.120	2.560	
4-8-5	80.133	68.772	3.322		**5-8-4**	86.222	3.469		**6-8-5**	78.037	3.053	
					5-8-5	72.231	2.481		**6-9-1**	78.573	2.772	
									6-9-2	80.415	2.859	
									6-9-3	76.360	2.751	
									6-9-4	80.130	2.930	
									6-9-5	81.776	3.192	
									6-9-6	82.727	3.151	

Series 3 – Effect of Stress Rate

NOTE – Specimen named in the following method: Batch # - Stress Rate - Sequence of Testing

Batch 7 (0.40 W/C)			Batch 8 (0.50 W/C)			Batch 9 (0.60 W/C)		
Specimen Name	Failure Stress (MPa)	Note	Specimen Name	Failure Stress (MPa)	Note	Specimen Name	Failure Stress (MPa)	Note
7-1-1	8.603		**8-1-1**	6.729		**9-1-1**	5.787	
7-1-2	7.837		**8-1-2**	6.470		**9-1-2**	5.814	
7-1-3	7.995		**8-1-3**	6.361		**9-1-3**	6.043	
7-1-4	5.878		**8-1-4**	4.271		**9-1-4**	*8.623*	*Leak failure*
7-1-5	*9.522*	*Leak failure*	**8-1-5**	*7.434*	*Leak failure*	**9-1-5**	4.548	
7-2-1	8.564		**8-2-1**	6.900		**9-2-1**	5.833	
7-2-2	8.608		**8-2-2**	5.680		**9-2-2**	*8.720*	*Leak failure*
7-2-3	6.599		**8-2-3**	6.412		**9-2-3**	*8.552*	*Leak failure*
7-2-4	7.690		**8-2-4**	6.764		**9-2-4**	6.617	
7-2-5	7.191		**8-2-5**	6.402		**9-2-5**	5.968	
7-3-1	6.743		**8-3-1**	7.276		**9-3-1**	6.341	
7-3-2	8.116		**8-3-2**	7.013		**9-3-2**	5.613	
7-3-3	7.338		**8-3-3**	7.731		**9-3-3**	6.297	
7-3-4	7.333		**8-3-4**	*8.500*	*Leak failure*	**9-3-4**	5.333	
7-3-5	7.157		**8-3-5**	7.068		**9-3-5**	5.366	
7-4-1	6.975		**8-4-1**	5.995		**9-4-1**	5.739	
7-4-2	6.230		**8-4-2**	6.584		**9-4-2**	6.772	
7-4-3	6.553		**8-4-3**	5.155		**9-4-3**	5.496	
7-4-4	6.664		**8-4-4**	4.912		**9-4-4**	5.490	
7-4-5	5.849		**8-4-5**	5.579		**9-4-5**	6.047	
7-5-1	7.658		**8-4-6**	6.466		**9-5-1**	5.774	
7-5-2	7.090		**8-5-1**	6.084		**9-5-2**	6.634	
7-5-3	8.190		**8-5-2**	5.852		**9-5-3**	5.874	
7-5-4	8.114		**8-5-3**	6.516		**9-5-4**	6.588	
7-5-5	8.205		**8-5-4**	6.121		**9-5-5**	5.615	
			8-5-5	6.315				
			8-5-6	5.790				

Series 4 – Effects of Creep

NOTE – Specimen named in the following method: Batch # - Stress-to-Strength Level - Sequence of Testing

Batch 10 (0.40 W/C)				Batch 11 (0.50 W/C)				Batch 12 (0.60 W/C)			
Specimen Name	Held Time (s)	Failure Stress (MPa)	Note	Specimen Name	Held Time (s)	Failure Stress (MPa)	Note	Specimen Name	Held Time (s)	Failure Stress (MPa)	Note
10-0-1	0.000	8.397	Strength	**11-0-1**	0.000	7.439	Strength	**12-0-1**	0.000	6.550	Strength
10-0-2	0.000	8.680	Strength	**11-0-2**	0.000	6.138	Strength	**12-0-2**	0.000	4.793	Strength
10-0-3	0.000	6.675	Strength	**11-0-3**	0.000	7.734	Strength	**12-0-3**	0.000	5.711	Strength
10-x-1	0.000	6.593	Premature	**11-x-1**	0.000	6.155	Premature	**12-x-1**	0.000	4.691	Premature
10-x-2	0.000	6.941	Premature	**11-x-2**	0.000	6.410	Premature	**12-x-2**	0.000	4.477	Premature
10-x-3	0.000	7.014	Premature	**11-x-3**	0.000	5.658	Premature	**12-86-1**	97.205	4.956	
10-x-4	0.000	6.539	Premature	**11-x-4**	0.000	6.333	Premature	**12-86-2**	58.502	4.962	
10-86-1	134.598	6.727		**11-x-5**	0.000	5.755	Premature	**12-86-3**	156.207	4.958	
10-86-2	277.916	6.758		**11-x-6**	0.000	6.175	Premature	**12-86-4**	0.891	4.958	
10-86-3	44.001	6.733		**11-x-7**	0.000	5.799	Premature	**12-86-5**	27.204	4.962	
10-86-4	100.685	6.762		**11-x-8**	0.000	9.081	Premature	**12-86-x1**	2080.196	4.962	Time Limit
10-86-x	877.636	*6.727*	*Leak*	**11-x-9**	0.000	8.082	Premature	**12-86-x2**	962.627	4.958	Leak
10-88-1	16.798	6.900		**11-x-10**	0.000	8.413	Premature	**12-88-1**	116.315	5.086	
10-88-2	34.705	6.900		**11-86-1**	62.207	5.771		**12-88-2**	182.005	5.095	
10-88-3	17.095	6.898		**11-86-2**	514.411	5.762		**12-88-3**	14.202	5.097	
10-88-4	194.897	6.900		**11-86-3**	771.641	5.767		**12-88-4**	74.614	5.101	
10-88-x	876.519	*6.900*	*Leak*	**11-86-x**	2410.460	5.701	Time Limit	**12-88-x**	762.925	5.099	Leak
10-90-1	56.596	7.066		**11-88-1**	23.891	5.907		**12-90-1**	53.907	5.221	
10-90-2	4.609	7.086		**11-88-2**	9.797	5.903		**12-90-2**	28.001	5.223	
10-90-3	142.895	7.221		**11-88-3**	49.708	5.910		**12-90-3**	7.391	5.217	
10-90-x	872.118	*7.196*	*Leak*	**11-90-1**	286.308	5.969		**12-90-4**	82.300	5.231	
10-92-1	70.690	7.266		**11-90-2**	53.813	6.042		**12-90-5**	88.799	5.227	
10-92-2	135.911	7.223		**11-92-1**	195.348	6.014		**12-92-1**	346.609	5.314	
10-92-3	143.004	7.221		**11-92-2**	63.293	6.110		**12-92-2**	337.414	5.316	
10-94-1	464.629	7.411		**11-92-3**	53.892	6.102		**12-92-3**	183.506	5.316	
10-94-2	49.205	7.374		**11-94-1**	1.703	6.303		**12-92-4**	156.503	5.314	
10-94-3	274.321	7.382		**11-94-2**	669.508	6.303		**12-94-1**	124.206	5.445	
10-94-4	163.599	7.377		**11-z-946**	89.614	6.534		**12-94-2**	154.615	5.444	
10-z-920	690.130	6.433		**11-z-990**	82.203	6.884		**12-94-3**	21.406	5.436	
10-z-942-2	125.504	7.041						**12-94-4**	242.007	5.438	
10-z-960	17.000	6.751						**12-94-5**	12.094	5.434	