BOND CHARACTERISTICS OF HIGH STRENGTH LIGHTWEIGHT CONCRETE

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OF
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CONCRETE

by

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ABSTRACT

Future construction of concrete floating platforms for offshore oil exploration and development off the East Coast of Canada may lead to a substantial increase in the use of high strength lightweight (HSLW) concrete in Canada. While HSLW concrete has been extensively used in other areas of the world such as Norway, its use to date in Canada has been limited. HSLW concrete with its improved durability and lightweight characteristics is a very much sought after material in the construction of concrete floating platforms. However, the efficient use of HSLW concrete in Canada is limited by the following two restrictions in the Canadian concrete design code A23.3 (2): first, in calculating the bar development length the maximum permissible value for the compressive strength of the concrete is limited to 64 MPa, secondly for lightweight concrete the minimum development length must be increased by 30%. The objective of this research was to determine the bond strength characteristics of 25 mm and 35 mm deformed reinforcement bars embedded in 80 MPa HSLW concrete and to assess whether or not the code restrictions are justified.

The experiment consisted of performing a total of 72 pullout and push-in test to evaluate the bond behavior under both monotonic and cyclic loading. The effect of tension and compression along with various rates of loading were investigated for the monotonic tests as well as changing the rate of loading for the cyclic tests. Each of the specimens was confined with 10 mm stirrups, representing a well-confined member that is typical in an offshore floating platform. The concrete used in the experiment had an average compressive strength of 83.1 MPa. The results were evaluated and compared to the work by other researchers on high strength normal weight (HSNW) concrete as well as to the Australian, American and Canadian design codes.

The results indicated that HSLW concrete behaves very similar to HSNW concrete and the maximum bond stress for HSLW concrete is greater than that of normal strength lightweight concrete. The bond stress versus displacement curve indicates a sharp nearly linear ascending portion of the curve followed by a steep descending portion indicating very brittle behavior, which is characteristic to high strength concrete. The cyclic tests indicated that cyclic loading does not have a significant effect on the bond strength provided that the maximum cyclic displacement is less than the peak load displacement in the monotonic test. A comparison of the test results to the various code equations indicates that the current codes are too conservative for HSLW concrete. In particular it is proposed that the concrete density modification factor for lightweight concrete in the Canadian design code be reduced from 1.30 to 1.10 for the case of HSLW concrete. Finally, it was determined that an expression based on the cubic root of the concrete compressive strength rather than the square root better describes the bond behavior of HSLW concrete.

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Table of Contents

Page

List of Tables

List of Figures

List of Figures (cont'd)

Page

Chapter 1

"There is also a kind of powder which from natural causes produces astonishing results .... This substance, when mixed with lime and rubble, not only lends strength to buildings of other kinds, but even when piers of it are constructed in the sea, they set hard under water. ....the water taken in makes them cohere, and the moisture quickly hardens them so that they can set into a mass which neither the waves nor the force of water can dissolve."

Vitruvius, c.50 B.C.

Introduction

1.1 Bond Strength of High Strength Concrete

Over the past several decades the development of concrete technology has resulted in a changing definition of high strength concrete. During the 1930's, Professor Hollister, Past President of the American Concrete Institute, predicted that the compressive strength of concrete would reach 70 MPa. The compressive strength of concrete available in the construction industry did increase over time. For instance, in the 1950's concrete with a compressive strength of 30 MPa was considered to be high strength, and by the end of the 1980's concrete strengths reached 135 MPa with concretes in the 70 to 80 MPa strength range being common place to the construction industry in 2001.

While high strength lightweight concrete has found its way into a variety of applications from high rise buildings to long span bridges, the primary area of interest for this research is for the use of high strength lightweight concrete in offshore floating platforms. The improved durability and lightweight characteristics has resulted in high strength lightweight concrete being used extensively in the construction of offshore concrete platforms for the North Sea. Certain portions of the topsides of the Hibernia GBS were also constructed using high strength lightweight concrete. The oil reserves off Newfoundland have been estimated to be 9 billion barrels. The Hibernia and Terra Nova projects are capable of developing just 1.5 billion barrels of the total reserve. This leaves huge potential for the construction and use of concrete floating platforms over the next several decades. If the research can relieve the current design code restrictions placed on high strength lightweight concrete, then floating concrete platforms may become cost competitive with steel platforms, thereby maximizing the employment benefits to Newfoundland and Labrador.

The principle reason for using high strength concrete is that it offers a cost effective solution to many design problems encountered in complex structures while at the same time providing higher strength and improved durability. Utilizing high strength concrete leads to a reduction in member sizes resulting in more efficient floor plans and aesthetically pleasing structures. Furthermore the use of high strength lightweight concrete stretches the design envelope by giving engineers the ability to span longer distances as the dead load of the structural member is drastically reduced. However, the design code limitations placed on high strength concrete stems from the fact that high strength concrete exhibits brittle behaviour under heavy loading, a very undesirable material characteristic.

The fundamental requirement of reinforced concrete design is that there be sufficient transfer of the tensile force in the reinforcement to the concrete. For the case of deformed reinforcement bars this force transfer is usually through a combination of adhesion of the concrete to the surface area of the bar, friction as well as bearing of the deformations directly upon the concrete. For plain bars only the adhesion and the friction contribute to the bond. This force, which prevents the longitudinal movement of the bar within the concrete, is referred to as the bond force. Bond stress results from a change in bond force along the length of the bar (1).

The length over which the full yield strength of the bar can be developed is known as the bar development length (1). In an effort to assist designers, the various concrete design code groups throughout the world have developed empirical equations that are used to determine the minimum bar development length. A series of modification factors are then applied to the minimum development length to account for the different situations that may affect the ultimate bond strength of the bar. The Canadian code CSA A23.3 (2) contains four modification factors. The location factor accounts for the casting position of the bar. The coating factor accounts for the reduction in bond strength due to the application of an epoxy coating that reduces the adhesion and friction between the bar and the concrete. The concrete density factor increases the development length for semi-low and low density concrete. The bar size factor decreases the minimum development length for bar sizes smaller than 20 mm.

1.2 Scope of Research

The scope of research performed as part of this thesis is to examine the bond strength characteristics of high strength lightweight concrete. The results of this research will assist with the assessment of the validity of the concrete density factor of 1.3 that is currently required under CSA A23.3 (2) in the development length calculation for lightweight concrete. The main objectives of the study are as follows:

- Determine the bond characteristics of high strength lightweight concrete under different monotonic loading parameters in tension and compression
- Determine the effect of rate of loading on the bond strength of high strength lightweight concrete.
- Determine the effect of cyclic loading on the bond strength of high strength lightweight concrete.
- Study the test results in efforts to more accurately determine the bond strength of high strength lightweight concrete.
- Compare the behavior of high strength lightweight concrete to that of high strength normal weight concrete.
- Compare the test results of high strength lightweight concrete to the calculated values using the design provisions of various design codes including CSA A23.3 (2).

The current design practices and CSA code equations were mainly developed using concrete with lower compressive strengths, thus the limitation for $f'_c \leq 64$ MPa.

These code provisions need to be reassessed based on more current research on high strength concrete. This research will provide test data on the bond performance of high strength lightweight concrete and will take us one step closer to better understanding the total performance characteristics of high strength lightweight concrete.

1.3 Outline of Thesis

The thesis consists of six chapters. It begins with an introduction of high strength lightweight concrete and an overview of the significance of the research. Chapter 2 provides a summary of the literature review that was undertaken as part of the work. Chapter 3 discusses the experimental test setup including the different parameters that will be investigated. Chapter 4 is a discussion of test results, including graphs and tables that show the affect of the various tested parameters. Chapter 5 includes a comparison of the test results on high strength lightweight concrete with the results by other researchers on high strength normal weight concrete. Furthermore, it compares the test results against the bond strength capacity as calculated by various design code authorities throughout the world, including the Canadian Code. The thesis concludes with Chapter 6 providing a summary and conclusion of test results along with recommendations for further study.

Chapter 2

Literature Review

2.1 Introduction

As designers strive to improve the structural performance of reinforced concrete structures such as long span bridges and offshore concrete platforms there has been significant interest in the use of high strength lightweight concrete (HSLW). HSLW concrete is the term given to concrete with a compressive strength exceeding 70 MPa and a unit weight ranging from 1600 kg/m^3 to 2000 kg/m^3. The use of HSLW concrete represents a 15 to 30% reduction in overall weight compared to normal weight concrete of the same compressive strength. As the use of HSLW concrete is relatively new, there is little published literature available on its structural behavior, especially in the area of the bond characteristics of HSLW concrete.

The basic principle of reinforced concrete design is that the concrete resists the compressive forces while the tensile forces are resisted by the steel reinforcement. The transfer of the tensile force from the concrete to the steel reinforcement results in the development of tangential stress components along the contact surface between the concrete and the reinforcement bar. The stress acting parallel to the bar along this interface region is known as the bond stress. Past research has shown that there are three main components that comprise the bond strength of reinforced concrete, which act to resist the bond stress: adhesion of the concrete to the reinforcement bar, frictional stress transfer and mechanical interlock. For plain reinforcement bars, only the adhesion and frictional components contribute to the bond strength. However, for deformed reinforcement bars, all three components of the bond strength are present, with the protruding ribs on the bar bearing against the concrete being the major contributor to the bond strength. It is because of the superior bond strength of deformed reinforcement bars that CSA A23.3 (2) states that only deformed bars shall be used as reinforcement, except that plain bars may be used for spirals and plain bars smaller than 10 mm may be used for stirrups or ties (1). Recent developments in the use of headed bar anchorages are being introduced to the various codes as a means to improving the bond strength of both plain and deformed reinforcement bars.

The superior performance of HSLW concrete is dependent on having adequate bond strength. The scope of the research is to examine experimentally the factors influencing the bond strength and to analytically predict the bond strength of deformed reinforcing bars in HSLW concrete when subjected to monotonic and cyclic loading. A comparison of the results with various research investigations will be presented in later discussions.

2.2 Bond Characteristics of High Strength Concrete

While much research has been performed on the bond characteristics of normal strength normal weight concrete, little has been done on high strength concrete, especially HSLW concrete. In addition, much of the research and development in the area of HSLW concrete was performed as part of technical studies associated with the construction of specific structures, mainly offshore concrete platforms. As a result much of the research pertaining to the structural behavior of HSLW concrete is confidential between the various research institutions and the oil companies.

The earliest reported bond research was by Abrams (3) which involved observing the behavior of both plain and deformed reinforcement in normal strength concrete. Later, Glanville (4) observed that bond failure occurred at higher stress levels for push type testing as compared with pullout testing.

Clark (5) used the pullout test to study the effect of various rebar patterns in normal strength concrete. It was concluded that the most effective deformation pattern was one where the shearing area is less than 10 times the deformation bearing area, with the shearing area defined as the perimeter of the bar multiplied by the deformation spacing. Clark also provided other guidelines such as the deformation spacing should not exceed 70 percent of the bar diameter, and the deformation height be a minimum 4% to 5% depending on bar size.

Primary and secondary cracks were observed in research by Mehlhorn and Kollegger (6). From studying the contact surface between the concrete and reinforcement steel they concluded complete compatibility between concrete and steel based on steel stress, bond stress and concrete stress.

From a series of 72 test specimens, Somayaji and Shah (7) developed an analytical model to predict the cracking response and tension stiffening in a reinforced concrete specimen subject to tension. The researchers concluded that the local bond stress slip relationship was nonlinear and not consistent at all sections throughout the length of the bar.

Shah et al. (8) reported that the microstructure of the interfacial zone between the aggregate and the cement paste greatly influences the mechanical properties such as stiffness, shear and bond strength. Processes involving aggregate pretreatment and improved mixing methods would significantly improve the mechanical properties of the interfacial region.

Hadje-Ghaffaei et al. (9) performed research on the bond characteristic of epoxy-coated reinforcement. The results of this work indicated that the bond strength is significantly reduced for epoxy-coated reinforcement. It was also observed that the lack of vibration usually present when high slump concrete is being pumped had a negative effect on the bond strength for both epoxy and uncoated reinforcement. These studies showed that the development length modification factor for epoxy coated reinforcement in the current ACI code is conservative and could be reduced from 1.5 to 1.35 for all sizes of bars.

In 1993, Darwin and Graham (10) studied the affect of deformation height and spacing on the bond strength of reinforcing bars based on a relative rib area approach. The results showed that by providing confinement either in the form of transverse reinforcement or additional concrete cover, the bond strength increased as the relative rib area was increased.

Hamad (11) concluded from a series of pullout specimens and beam specimens under positive bending that the bond capacity depends on the rib face angle, rib spacing and rib height. A comparison of various rib angles resulted in a rib face angle of 60 degrees providing the best load slip performance and highest ultimate bond strength. In addition, rib spacing equal to 50% of bar diameter and a rib height equal to 10% of the bar diameter was the optimum rib configuration.

Esfahani and Rangan (12) carried out testing to determine the effects of rib face angle on the bond capacity in high strength concrete. The results indicated that the bond strength of bars with rib face angles between 23 and 27 degrees is significantly lower than bars with rib face angles between 40 and 47 degrees.

Darwin et al. (13). studied the splice strength of bars with a high relative rib area. The test concluded that the splice strength of uncoated reinforcement increased as the relative rib area increased provided there was adequate transverse reinforcement. The tests also showed a 20% reduction in the development length modification factor for epoxy-coated reinforcement.

Azizinamini et al. (14) and Azizinamini. et al. (15) closely examined the bond performance and tension development length of reinforcement steel in high strength concrete. From these tests it was concluded that increasing the development length was not the most efficient way to increase bond capacity in high strength concrete. Furthermore. it was proposed that a minimum stirrup requirement be implemented into design of splice regions.

Hwang et al. (16) experimentally investigated the bond strength of deformed reinforcement bars in high strength concrete. The equivalent relationship between the tensile splice length and tensile development length was presented. This study concluded that the bond performance of high strength concrete without silica fume was similar to normal strength concrete, and that the presence of silica fume caused a decrease in the bond strength of deformed reinforcement.

The work of Azizinamini et al. (17), concluded that the current ACI limitation of 100 psi on $\sqrt{f'_c}$ for the calculation of tension development length is not justified provided that there is sufficient transverse reinforcement. From their work, it was shown that the addition of transverse reinforcement is equivalent to increasing the stress in a developed bar. Furthermore, the minimum amount of stirrup proposed was determined based on 104 MPa concrete. It was suggested that the amount of transverse reinforcement be decreased linearly as the concrete strength decreases.

2.3 Properties of lightweight aggregate concrete.

2.3.1 General

With increased use of structural lightweight aggregate concrete during the 1950's, more emphasis was being placed on recommendations for structural design, including methods for consideration of shear in beams and frames. ACI Committee 213, Properties of Lightweight Aggregate Concrete, was organized in 1946 and assumed the responsibility for code developments with regards to the use of structural lightweight concrete. Structural lightweight concrete is generally considered to be concrete with compressive strength in excess of 17.5 MPa and unit weight of 1950 kg/m^3 or less (18). However, in recent decades, high strength lightweight aggregate concrete (HSLW) has gained popularity, especially among those designers who strive to design and construct structures from concrete that would traditionally be built in structural steel, such as long span bridges and floating oil platforms.

With this increased use of HSLW concrete, it is of the utmost importance that we understand the mechanical properties of HSLW concrete. The remaining sections of this chapter summarize the available research with respect to the mechanical properties of HSLW concrete. More specifically it will introduce the current state of knowledge as proposed by the Eurocode (19) for concrete structures referred to hereafter as EC-2. The intention is for the EC-2 (19) draft to become the standard for the design of structures to be used throughout Europe by 2003.

2.3.2 Stress-strain behavior in uniaxial compression

High strength lightweight concrete exhibits significantly different properties from that of normal-strength lightweight and of high-strength normal weight concrete as reported by Martinez et al. (20). Typical axial stress-strain relationship for moist-cured lightweight concrete is shown in Figure 2.1. It was shown that as the strength of lightweight concrete increases, the stress-strain curves become steeper and the stress stain relationship is nearly linear. The stress strain characteristics as reported by Hoff (21) are shown in Fig 2.2. The behavior shown is typical for HSLW concrete where the initial slope is reduced compared with normal weight concrete. Also, the steep and limited descending portion is indicative of the low post cracking ductility of HSLW concrete. The maximum permissible strain as per the ACI 318 (22) code is 0.003, which is in agreement with the reviewed research.

2.3.3 Modulus of elasticity

Research has shown the modulus of elasticity for lightweight concrete to be lower than that of normal weight concrete. At 40 percent of ultimate stress the values of modulus of elasticity ranged from 18 to 30 GPa for concretes having compressive strengths ranging from 55 to 82 MPa (18, 19 & 20). The type of aggregates as well as aggregate volume and stiffness have a large effect on both the compressive strength and modulus of elasticity of lightweight concrete. The ACI 318 (22) code expression for predicting the modulus of elasticity for lightweight concretes with compressive strengths less than 41 MPa overestimates the modulus of elasticity for lightweight concrete with compressive strengths exceeding 41 MPa. The current state of knowledge as presented in the EC-2 report (19) states that the material properties of lightweight aggregate concrete can be derived using conversion factors applied to the properties of normal weight concrete.

Martinez et al. (20) proposed an equation that better fit the experimental data for most densities of lightweight concrete.

$$E_c = (3320 \sqrt{f'_c} + 6900)(w_c/2320)^{1.5} \quad \text{MPa} \qquad \textbf{(2.1)}$$

where w_c is the unit weight of the concrete in kg/m^3

For normal weight concrete ($w_c \geq 2320\ kg/m^3$) with

$21 < f'_c < 83$ MPa

and for lightweight concrete ($1440 < w_c < 2320\ kg/m^3$) with

$21 < f'_c < 62$ MPa

This was confirmed by Hoff (21) who reported that the ACI equation overestimated the modulus of elasticity in the range of 9 to 30 percent, while equation 2-1 produced a modulus elasticity values to within 5 percent of the experimental data. Zhang and Gjorv (23) proposed another equation for calculating the modulus of elasticity based on the square of the cube compressive strength as follows:

$E_c = 1.19\ F_{cu}^{2}$, where F_{cu} = cube compressive strength **(2.2)**

2.3.4 Poisson's Ratio

Poisson's ratio is the ratio of the transverse strain to the axial strain under uniform axial stress. Martinez et al. (20) used the ASTM procedures to determine the values for Poisson's ratio from 100 mm x 204 mm lightweight concrete cylinder tests. While all specimens had a compressive strength of 62 MPa, the values of Poisson's ratio varied slightly depending on curing method, ranging from 0.13 to 0.24 for moist-cured concrete, and 0.15 to 0.26 for dry-cured concrete. Hoff (21) reported a different trend for concrete in the 55 to 72 MPa range. The value of Poisson's ratio decreased for concrete exposed for air drying with values in the range of 0.21 to 0.23 for moist-cured and 0.16 to 0.17 when the concrete was exposed for additional air drying. The average value of Poisson's ratio was determined 0.20 regardless of compressive strength, curing conditions and age of lightweight concrete by Martinez et al. (20), Shilder (25) and Carrasquillo et al. (26).

2.3.5 Tensile Splitting Strength

Traditionally, the tensile splitting strength for lightweight concrete is expressed as a coefficient times the square root of the compressive strength. Martinez (20) reported values for tensile splitting strength of lightweight concrete. For both moist-cured and dry-cured, the small coefficients tend to apply to the higher strength concretes, while the larger ones apply to those of lower strength. The recommended value for tensile splitting strength is taken as:

$$f_{sp} = 0.42\ \sqrt{f'_c}\quad , \text{MPa} \tag{2.3}$$

$$21 < f'_c < 62$$

This proposed expression is consistent with the ACI 318 (22) expression. Hoff (21) reported values for the tensile splitting strength (f_{sp}) of semi (sand) lightweight concrete that ranged from $0.43\sqrt{f'_c}$ to $0.49\sqrt{f'_c}$ MPa or greater for moist-cured concrete for compressive strengths ranging from 55 to 72 MPa. Most of the researchers agreed that the effects of drying have a great influence on the value of the splitting tensile strength. Other investigators (18, 22, 25) reported values of f_{sp} that ranged from $0.47\sqrt{f'_c}$ to $0.6\sqrt{f'_c}$ MPa for compressive strengths in the range of 53 to 100 MPa.

2.3.6 Modulus of Rupture

Many researchers(20, 21, 23, 27, 28) have investigated the modulus of rupture f_r of lightweight concrete and have determined the values to be in the range of $0.2\sqrt{f'_c}$ to

$0.54\sqrt{f'_c}$ MPa for dry-cured concrete and from $0.51\sqrt{f'_c}$ to $0.91\sqrt{f'_c}$ MPa for moist-cured concrete. It was concluded from this research that curing conditions have an effect on the values of modulus of rupture. Martinez et al. (20) reported that the ACI 318 (22) value of $0.46\sqrt{f'_c}$ MPa overestimates the value of the modulus of rupture and that $0.35\sqrt{f'_c}$ MPa is a more accurate value for dry-cured lightweight concrete. However, the ACI 318 (22) expression can be used to predict the modulus of rupture of moist-cured high strength lightweight aggregate concrete.

2.3.7 Direct Tension Strength

Fig. 2.3 shows a comparison between the descending branches in tension for normal weight concrete and lightweight concrete. The graphs indicate that a considerable part of fracture energy is consumed at relatively high stress levels for lightweight aggregate, while for normal weight concrete a significant amount of energy is consumed at low stress and relatively large crack widths. This may explain why lightweight concrete sometimes behaves with more ductility than would be expected. Markeset and Hansen (29) reported a study of the tension properties of lightweight concrete in which the lightweight aggregate concrete was found to be considerably more brittle than normal weight concrete. In spite of these results, the ratio of flexural strength to tensile strength for lightweight concrete was taken to be similar to that of normal weight concrete.

2.3.8 Unit Weight

The unit weight of lightweight aggregate concrete with compressive strength in excess of 45 MPa. ranges from 1600 kg/m^3 to 2000 kg/m^3. This unit weight represents a weight reduction of 15 to 30 percent compared to normal weight concrete of the same compressive strength.

2.3.9 Strength Gain with Age

An attractive property of high strength lightweight aggregate concrete is that it obtains 92 % of the 28 day strength in 7 days (21). While the initial strength gain in high strength concrete is more than normal weight the difference in strength gain becomes negligible at later ages (20). Fig 2.4 shows the average rate of strength development for lightweight concretes for test data. While higher temperatures often improves the 7 day strength it appears to cause a reduction in the 28 day strength as compared to samples not subjected to high temperatures during the initial curing period. It is noted in the EC-2 (19) report that the increase in strength after 28 days is less for lightweight concrete than normal weight concrete. Another European Union report (30) reported that for lightweight concrete the sustained loading effect is more pronounced than for normal aggregate concrete

2.3.10 Creep

Hoff (21) reported test results on high strength lightweight concrete according to the ASTM C-512 (31). The observed creep strain ranged from 0.228 to 0.435 at 90 days, under compressive stress of 6.9 MPa at 22.8°C. As with normal density concretes, the rate of creep was observed to decrease with time. The creep strain of the higher strength concrete was less than that for the concrete of lesser strength. Leming (18) reported an average value of specific creep at one year to be 0.24 microstrain/psi for high strength lightweight concrete, slightly higher than that of the normal weight concrete. The specific creep of high strength lightweight concrete was found to be within the range of values provided for structural design by ACI Committee 209. Shideler (25) has reported ultimate creep microstrain/psi to be 0.545 for 48 MPa concrete and 0.52 microstrain/psi for 62 MPa lightweight concrete.

The EC-2 report (19) states that the creep coefficient ϕ, can be assumed to be equal to the value of normal density concrete multiplied by a factor equal to:

$$(\rho/2400)^2 \quad \text{when } \rho > 1800 \text{ kg/m}^3 \text{ and} \tag{2.4}$$

$$1.3\,(\rho/2400)^2 \quad \text{when } \rho < 1500 \text{ kg/m}^3 \tag{2.5}$$

For intermediate values of ρ, linear interpolation may be used. While this EC-2 (19) report along with other standards including the Norwegian (NS3473), Japanese (JSCE), and German Code (DIN4219), all give similar formulations. Work by Kordina (32) and Neville (33) cast doubts on the correctness of the statement that creep of light weight concrete is smaller than that of normal weight concrete. Kordina (32) suggests that creep is a function of cement paste and not the aggregate. This would mean that lightweight and normal density concrete of the same paste compositions should have the same creep. Neville (33), on the other hand, distinguishes between the paste and aggregate as two load-carrying components. Since most lightweight aggregates have a lower stiffness than normal weight aggregates, the paste will carry more stress and hence, the creep should be higher in lightweight aggregate concrete.

2.3.11 Shrinkage

Bilodeau et al. (27) reported values of total drying shrinkage strain, after 448 days of drying, for concrete with compressive strength between 46 and 72 MPa, to range from 518 to 667 microstrain. The total weight loss of the concrete after 448 days of air-drying ranged from 1.53 to 5.41 percent. Hoff (21) reported that the concretes from the mixture containing silica fume experienced less shrinkage than those containing fly ash and slag. Fig. 2.5 shows the drying shrinkage versus age for the three mineral admixtures. Shideler (25) reported one-year shrinkage values of 545 and 512 microstrain for 48 MPa and 62 MPa concrete strength respectively.

The EC-2 report (19) separates shrinkage into two parts: drying shrinkage and autogeneous shrinkage as represented in the formulas for final shrinkage strain:

$$\epsilon_{cs} = \epsilon_{cd} + \epsilon_{ca} \tag{2.6}$$

where:

ϵ_{cs}= final shrinkage strain

ϵ_{cd} = drying shrinkage strain

ϵ_{ca} = autogeneous shrinkage

The drying shrinkage values for lightweight concrete can be obtained by multiplying the values for normal density concrete with a factor η_3 defined by:

$$\text{LC12/15 to LC/20} \quad \eta_3 = 1.5 \tag{2.7}$$

$$\text{LC20/25 and higher} \quad \eta_3 = 1.2 \tag{2.8}$$

where LC12/15 is the way that the different strength classes are defined. The LC means lightweight concrete, the first number is the characteristic cylinder strength and the second is for the characteristic cube strength.

The autogeneous shrinkage for Lightweight Aggregate Concrete (LWAC) is not clearly defined, but it will be considerably reduced if the aggregate is fully or partially saturated with water. As this is usually the case for LWAC, the autogeneous shrinkage of LWAC is neglected. Furthermore the contribution of autogeneous shrinkage decreases considerably with increased concrete strength.

2.3.12 Freeze and Thaw

Hoff (21) reported the test results for the freezing and thawing of concrete according to the ASTM C-666 Procedure A. for high strength lightweight concrete specimens. Fig. 2.6 shows the values of the relative dynamic modulus for three different mixes using silica fume, fly ash, and slag having compressive strength 56, 62 and 73 MPa respectively. In general, all of these concretes exhibited excellent performance.

2.3.13 Thermal Properties

Hoff (21) reported on the thermal properties of high strength lightweight concrete including thermal expansion, thermal conductivity, specific heat and thermal diffusivity. The study indicated that the result obtained for high strength lightweight concrete is favorably comparable to those of other concrete of similar densities. The EC-2 report (19) uses the same coefficient of thermal expansion for both normal density and lightweight concrete.

2.3.14 Temperature Development

Hoff (21) reported the results of a study of temperature development due to hydration. The maximum temperature rise in large sections of concrete made with high strength lightweight aggregate varied from 56 to 63 °C. Significant thermal gradients were developed in the concrete due to the better isolation characteristics of the lightweight aggregates. The use of mineral admixtures did not significantly influence the temperature regime. Peak temperatures were delayed only 4 to 8 hours with those materials.

2.3.15 Permeability

Zhang and Gjorv (23) reported that the permeability of high strength lightweight concrete of compressive strength 50 to 100 MPa appears to be very low, but it might be higher than that of normal weight concrete at a similar strength level. The use of natural sand instead of lightweight sand reduced the permeability.

2.3.16 Fatigue Strength

Fatigue strength of lightweight aggregate concrete is generally smaller than normal weight concrete. However, Leming (18) mentioned in his study that the high-strength lightweight concrete could have improved fatigue load resistance compared with conventional strength, normal weight concrete due to improved strain compatibility, similar elastic response, bond, and lack of bleeding of high strength lightweight concrete. Kojima et al. (34) confirmed that high strength lightweight concrete had similar fatigue characteristics to normal weight concrete both in air and underwater conditions. Kojima et al. (34) also confirmed that the compressive fatigue strength is influenced not only by the surrounding environment and strength of concrete but also by quality of lightweight aggregate. Aggregate characteristics such as absorption and strength need to be taken into consideration.

2.4 Design Considerations

The EC-2 (19) report also presented guidelines for assessing the bearing capacity of structures in the ultimate limit states. More specifically it addresses the behavior of lightweight concrete with respect to shear and punching as lightweight concretes are suppose to have smoother cracks than normal weight concrete. In normal weight concrete the cracks propagate around the aggregate whereas in lightweight the crack intersects the aggregate as lightweight aggregate have lower strength than gravel aggregate. The EC-2 (19) report concluded for members without shear reinforcement the shear capacity for lightweight concrete should be reduced by multiplying the shear equation for normal density concrete by a density factor η_l:

Where $$\eta_l = 0.40 + 0.60\rho/2400 \qquad \textbf{(2.9)}$$

and ρ is the upper limit of the oven dry density.

Also the coefficient in the shear equation of EC-2 (19) should be reduced from 0.12 to 0.10 to account for the use of lightweight concrete.

In members with shear reinforcement the variable inclination truss analogy that is used for normal weight concrete can also be used for lightweight concrete, including the same lower limit of the strut angle. The efficiency coefficient that is explained in the EC-2 (19) report must be reduced by 25% for lightweight concrete.

For members subject to punching shear the corresponding equation for normal weight concrete should be multiplied by the density factor determined in Equation 2.9. No further modification of the coefficients of the equations is required for lightweight concrete.

The EC-2 (19) report also suggest that the same principle be followed for torsion as for shear as the allowable compressive stress for lightweight aggregate concrete is dependent on both the density and the additional reduction factor for prismatic struts.

Moe et al. (35) reported that when detailing with lightweight aggregate concrete the minimum concrete cover requirements must be increased by 5 mm to be 50 mm for reinforcement steel and 60mm for prestressing steel.

Faust et al. (36) concluded that the type of matrix greatly influences the transverse behaviour much more than the longitudinal behaviour. The use of lightweight sand results in a stress strain relationship that is nearly linear up to failure load. However, using natural sand resulted in large transverse strains at approximately 80% of the compressive strength.

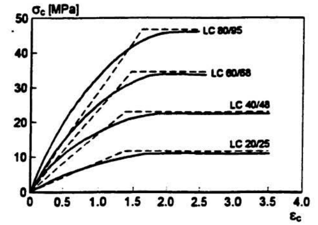

Figure 2.1 – Typical axial stress – strain for moist cured HSLW concrete (19).

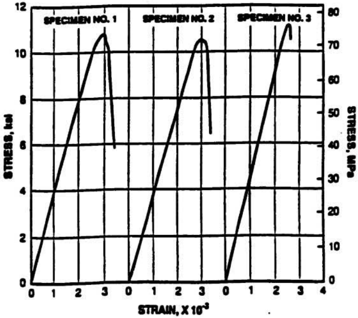

Figure 2.2 – Stress versus strain for HSLW concrete (21).

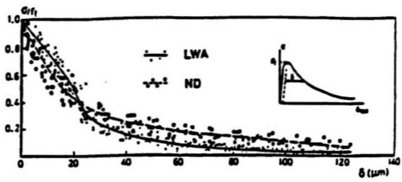

Figure 2.3 – Descending branches in tension for ND and LWA concrete (29)

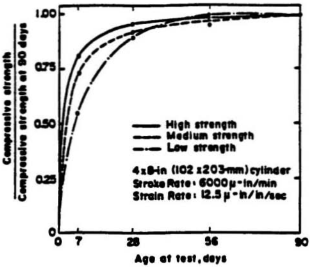

Figure 2.4 – Compressive strength gain with age for moist cured LWA concrete (20)

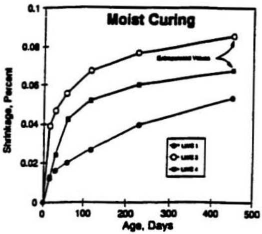

Figure 2.5 – Drying shrinkage versus age relationship (21)

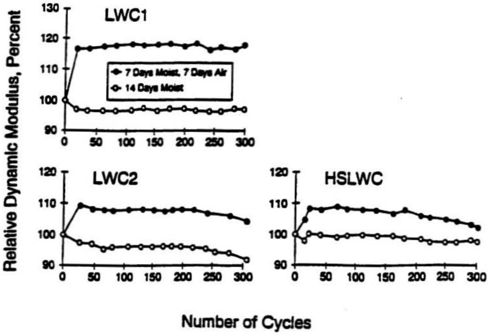

Figure 2.6 – Freezing and thawing behavior for LWA concrete (21)

Chapter 3

Experimental Investigation on the Bond Behavior of High Strength Lightweight Concrete

3.1 Introduction

The review of past research work presented in the previous chapter concluded that the following are the main factors in determining the bond characteristics of a deformed bars embedded in concrete: the effect of load history, confining reinforcement, rebar diameter, concrete strength, rebar spacing and rate of pull out. While other factors such as amount of concrete cover, aggregate size, reinforcement coatings and casting position can affect the bond strength it was not included in the scope of work for this thesis.

The remaining sections of this chapter will provide an overview of the materials used in the construction of the test specimens, the configuration of the test specimens, the test setup, including instrumentation and data acquisition, as well as the experimental test program.

3.2 Materials

Over the past couple of decades many researchers including Marzouk and Chen (38), Marzouk and Dajui (39), Marzouk and Hussein (40), Malhotra et al. (41) and Hoff (42 & 43) have made significant contributions to the material characteristics of high strength concrete. The material selection is extremely important if one is to take full advantage of its beneficial attributes such as resistance to freezing and thawing, chloride ion penetration and increased durability as it is commonly exposed to harsh and aggressive environments such as chemicals and salt water.

The concrete mix design used in this experimental program was developed by Osman and Marzouk (44) during their work on the behavior of HSLW concrete flat slabs under static and cyclic loading. The proportions to produce 1.0 m^3 of HSLW concrete for the experimental portion of this research are shown in Table 3.1.

The entire test specimens were constructed using Normal Portland Type 10 cement as produced by North Star Cement in Newfoundland. Mineral admixtures such as silica fume play a key role in the development of high strength concrete. The silica fume used in these test specimens had a specific gravity of 1.34 and a surface area of 200000 cm^2/gm, which is approximately 50 times finer than Portland cement. It is this fineness that results in more surface area for cement hydration and produces a much denser microstructure and higher strength concrete. However, the use of silica fume increases the demand for water in fresh concrete.

The increased demand for water is offset by the use of chemical admixtures such as superplastizers and water reducing retarders. The superplastizers significantly reduce the amount of water required and increases the slump while the retarder extends the initial set period to allow adequate time for placement. To maintain the water cement ratio of 0.29, superplastizer (Eucon 37), and retarder (TCDA Type DX), supplied by Euclid Admixture Canada Inc., were added to the mix in the amounts shown in Table 3.1.

The shape and coarseness of the fine aggregate determine the amount of absorption and therefore affect the amount of water required in a given mix design. Finer sands have more surface area and therefore more absorption for a given weight of material. The sand used in the mix was natural river sand that was obtained locally. The specific gravity of the sand was 2.73 and absorption of 0.42%.

The quality of the coarse aggregate greatly influences the quality and strength of the concrete. The strength of the aggregate, the adhesion of cement paste to the aggregate and the absorption characteristics are very important in the mix design. The coarse aggregate for this experiment consisted of a lightweight aggregate that was imported from North Carolina, USA under the trade name Stalite. It consists of a rotary kiln dried high quality slate. The maximum size of this lightweight aggregate was 19 mm with a specific gravity of 1.45 and a dry density of 960 kg/m^3.

The bar sizes used in the experiment was 25 mm and 35 mm. These bars along with the 10 mm stirrups were supplied from a local company and conformed to CSA Grade 400.

The mixing water for the specimens was from the municipal water supply for the City of St. John's. The water cement ratio for all batches was 0.30.

The concrete was batched over a two-day period using the 0.1 m^3 capacity drum mixer. Two cylinders were taken from the first day and one cylinder from the second day. These cylinders, along with the test specimens, were covered in polyethylene and moist cured for 28 days.

3.3 Test Specimens

The experimental program consisted of casting 36 specimens each for the two bar sizes, 25mm and 35mm rebar. To facilitate a comparison between the test results and the previous work done on high strength normal weight and normal strength normal strength concrete, the size of the specimen was the same as that used by Alavi-Fard and Marzouk (37), who modeled their setup after work by Eligehausen et al. (45) on normal strength concrete.

All of the specimens had a depth of 250 mm, however, the length and thickness of the specimens were a function of the bar diameter as indicated in Figure 3.1 and Table 3.2. The length of each specimen was set at 15 d_b while the thickness varied from 5 d_b to 7 d_b.

The horizontal casting position was chosen for all specimens with the bars positioned in the middle of the concrete section. The observed bond strength was expected to be superior to that of top or bottom placed bars. A 38 mm concrete vibrator was used to vibrate the concrete.

In efforts to ensure that the load to cause pullout of the embedded bar was less than the ultimate tensile strength of the bar, the bond length for each of the two bar sizes was 100 mm. This bond length was chosen to be consistent with the work of Alavi – Fard and Marzouk (37) and was intended to be representative of the bond stress. However, 100 mm proved to be sufficient bond length to develop the ultimate capacity of the threaded sections of some of the 25 mm specimens. The remaining sections of bar were covered with a PVC pipe with the ends of the pipes covered with plastic to prevent any concrete from entering the pipe sleeves, effectively increasing the bond length.

Special care had to be taken to prevent the movement of the pipe sleeves during concrete placement, resulting in an increase or decrease in bond length. By ensuring that the combined length of the two pipe sleeves used to cover the bar equaled the total length of the concrete specimen the actual bond length could be easily controlled. The total projection of the pipes from each end of the specimen equaled the actual bond length within each specimen. Figure 3.1 and Table 3.2 show the measurements used to calculate the actual bond length for each specimen that was tested.

3.3.1 Strain Gauges

Five samples of each bar size had electric strain gauges both attached to the bar and embedded in the concrete. The strain gauges were attached to a prepared section of the bar that was embedded in concrete. Preparing the section of bar involved grinding smooth approximately 30 mm length of bar. The surface was then hand sanded using progressively finer sand paper. The surface was cleaned using an etching solution to ensure good contact with the bar and the strain gauge was secured to the bar using an adhesive tape. To ensure the strain gauge did not become damaged during concrete placement it was covered with a waterproofing product known as M-coat and securely wrapped in electrical tape. The most favorable method to measure bond strain involved attaching the strain gauge to precast blocks as per Figure 3.2. These precast blocks, with the strain gauges attached, were placed adjacent to the bar and held temporarily in place using tie wire. The photograph in Figure 3.3 shows the placement of both types of strain gauges. The photographs in Figures 3.4 to 3.7 show the various stages of specimen construction.

3.4 Selection of Test Set-up

A structural steel frame at the structures lab of Memorial University of Newfoundland was used for the experimental programs. The frame was designed, fabricated and erected as part of previous experimental work at the Structures Laboratory at Memorial University of Newfoundland on high strength normal weight concrete.

3.4.1 Test Set Up

The testing frame consisted of two vertical W-shaped steel columns connected near the top by two steel channels bolted to the columns. The bases of the columns were bolted to the structural floor of the laboratory. The dimensions and configuration of the test frame is shown in Figure 3.8. Figure 3.9 shows a photograph of the test set up. It was equipped with an electro-hydraulically controlled testing actuator capable of applying loads of $\pm$ 690 kN (150 kips).

The instrumentation for the test setup is shown in schematic format in Figure 3.10 and in the photograph of 3.11. It essentially consists of an actuator with a load cell attached to measure the load being applied. The movement at the loaded end of the bar was measured using the linear variable differential transducer (LVDT) built into the actuator, whereas the movement at the free end of the bar was measured using a linear potential differential transducer (LPDT) which was mounted externally using a magnetic mounting apparatus.

The load being applied and the corresponding displacements, along with the strain gauge readings, were continuously scanned and recorded by the data acquisition system in addition to being displayed on the monitor.

Each one of the bars used in constructing the specimens has a standard coarse thread on one end of the bar. This was the means by which the specimen was connected to the yoke of the ram of the actuator.

3.5 Experimental Test Program

An overview of the test program for each of the two bar sizes tested is outlined in Table 3.3. The tests were subdivided into four categories for each bar size as follows:

- Monotonic Load in Tension
- Monotonic Load in Compression
- Rate of Pullout for Monotonic Load
- Rate of Loading for Cyclic Load

3.5.1 Monotonic Load in Tension

Under the monotonic load in tension series, the bars were loaded in tension by securing the yoke of the ram to the bar using a plate and nut. The specimen was preloaded to ensure proper seating of the specimen against the bearing plate. The LVDT was attached to the end of the bar protruding from the bottom of the specimen and the initial LVDT reading was recorded. The tensile load was stroke controlled and applied at a standard rate of 1.50 mm/min. The test was terminated after the load peaked and descended to near zero. In some cases the 100 mm bond length was sufficient to develop the ultimate capacity of the threaded section of the 25 mm bars and the bars broke on the ascending portion of the loading sequence.

3.5.2 Monotonic Load in Compression

The monotonic load in compression test was very similar to the tension test except that the ram pushed on the bar with the bottom surface of the specimen bearing against a bottom plate. The specimen was preloaded to ensure proper seating and the initial reading on the LVDT attached to the lower end of the bar was recorded. The compressive load was applied at the standard rate of 1.50 mm/min and the test was terminated after the load peaked and descended to near zero. Some of the threaded ends of the bars had to be shortened to prevent premature buckling of the bars under compression.

3.5.3 Rate of Loading

The third series of tests, involved the same test setup as monotonic load in tension, but varied the rate at which the tension load was applied. In addition to testing the standard 1.50 mm/min loading rate, the loading rate was increased 50 times to 70 mm/min and decreased 10 times to 0.150 mm/min. While the test using the 70 mm/min-loading rate only lasted a few seconds, testing at 0.150 mm/min took several hours to complete.

3.5.4 Cyclic Loading

The last group of tests involved cyclic testing using the three stages of loading described above. It was important to position the specimen such that the testing ram did not run out of stroke during the cyclic testing. The bottom plate was raised and the top plate was adjusted down to clamp the specimen between the top and bottom plates using the nuts on the four vertical support bars. The cycles were governed by setting the stroke control function to $\pm$ 3.75 mm for the first 10 cycles and $\pm$ 7.50 mm for the last five cycles.

Table 3.1 – Concrete Mix Proportions

High-Strength Lightweight Concrete (1.0 m^3)	
Cement	430 kg
Silica Fume	55 kg
Lightweight aggregate	590 kg
Sand	810 kg
Water cement ratio	0.30
Superplasticizer	7525 ml
Retarder	2300 ml

Table 3.2 - Specimen Dimensions (refer to Figure 3.1)

Dimension (mm)	D	A	B	C	E	F	G
Specimen	25	375	150	250	E	F	E+F
	35	525	175	250	E	F	E+F

Table 3.3 – Experiment Program

Series	Specimen Number	Investigated Parameter	Loading History	Number Of Cycles	Bar Number	Concrete Strength f'_c (MPa)	Vertical Confining Bars Diam.	Slip Rate mm/min	Bar Deformation Pattern
1	HSLW250	Loading History	Monotonic in Tension	0	25	83.1	10	1.50	Standard
	HSLW251			0					
	HSLW252			0					
	HSLW253			0					
	HSLW254			0					
	HSLW255			0					
	HSLW256			0					
	HSLW2514			0					
	HSLW2515			0					
	HSLW2516			0					
	HSLW25b1			0					
	HSLW25b2			0					
	HSLW2520		Monotonic in Compression	0	25	83.1	10	1.50	Standard
	HSLW2521			0					
	HSLW2522			0					
	HSLW2523			0					
	HSLW2524			0					
	HSLW2525			0					
	HSLW2526			0					
	HSLW2527			0					
	HSLW2528			0					
	HSLW25b5			0					

Table 3.3 – Experiment Program (continued)

Series	Specimen Number	Investigated Parameter	Loading History	Number Of Cycles	Bar Number	Concrete Strength f'_c (MPa)	Vertical Confining Bars Diam.	Slip Rate mm/min	Bar Deformation Pattern
2	HSLW258	Rate of Loading	Monotonic	0	25	83.1	10	1.50	Standard
	HSLW2513			0				1.50	
	HSLW257			0				75	
	HSLW2510			0				0.150	
	HSLW2511			0				0.150	
	HSLW2517			0				0.150	
	HSLW2518		Cyclic	10	25	83.1	10	1.50	Standard
	HSLW2519			10				1.50	
	HSLW25b4			10				1.50	
	HSLW2512			10				75	
	HSLW			10				.150	

Table 3.3 – Experiment Program (Continued)

Series	Specimen Number	Investigated Parameter	Loading History	Number Of Cycles	Bar Number	Concrete Strength f'c (MPa)	Vertical Confining Bars Diam.	Slip Rate mm/min	Bar Deformation Pattern
3	HSLW35a0	Loading History	Monotonic in Tension	0	35	83.1	10	1.50	Standard
	HSLW35a1			0					
	HSLW35a2			0					
	HSLW35a3			0					
	HSLW35a4			0					
	HSLW35a5			0					
	HSLW35a7			0					
	HSLW35a8			0					
	HSLW35a9			0					
	HSLW3511			0					
	HSLW35b0			0					
	HSLW35b1			0					
	HSLW3513		Monotonic in Compression	0	35	83.1	10	1.50	Standard
	HSLW3514			0					
	HSLW3515			0					
	HSLW3516			0					
	HSLW3517			0					
	HSLW3518			0					
	HSLW3519			0					
	HSLW3520			0					
	HSLW3521			0					
	HSLW3522			0					
	HSLW35b2			0					
	HSLW35b3			0					

Table 3.3 – Experiment Program (Continued)

Series	Specimen Number	Investigated Parameter	Loading History	Number Of Cycles	Bar Number	Concrete Strength f'c (MPa)	Vertical Confining Bars Diam.	Slip Rate mm/min	Bar Deformation Pattern
4	HSLW352	Rate of Loading	Monotonic	0	25	83.1	10	1.50	Standard
	HSLW355			0				1.50	
	HSLW35a6			0				75	
	HSLW356			0				0.150	
	HSLW357		Cyclic	10	25	83.1	10	1.50	Standard
	HSLW358			10				1.50	
	HSLW35b5			10				1.50	
	HSLW359			10				75	
	HSLW3510			10				.150	

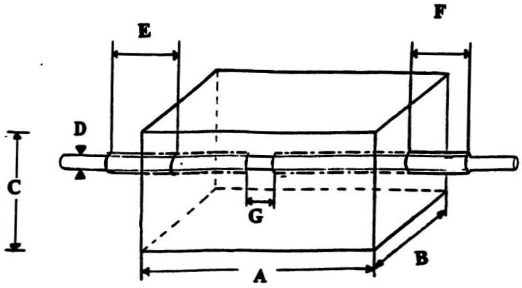

Fig. 3.1 Specimen measurements (refer to Table 3.2)

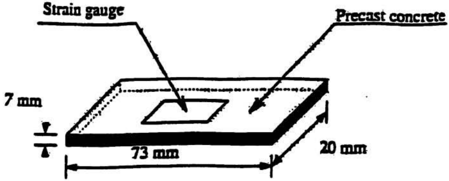

Fig. 3.2 Typical precast concrete with strain gauge

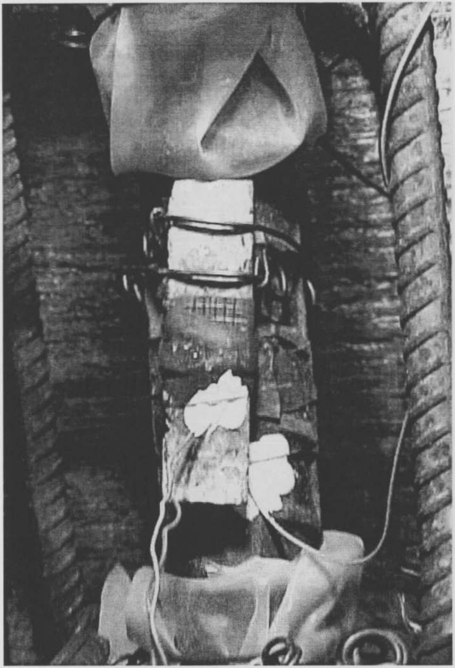

Figure 3.3 – Placement of concrete and steel strain gauges

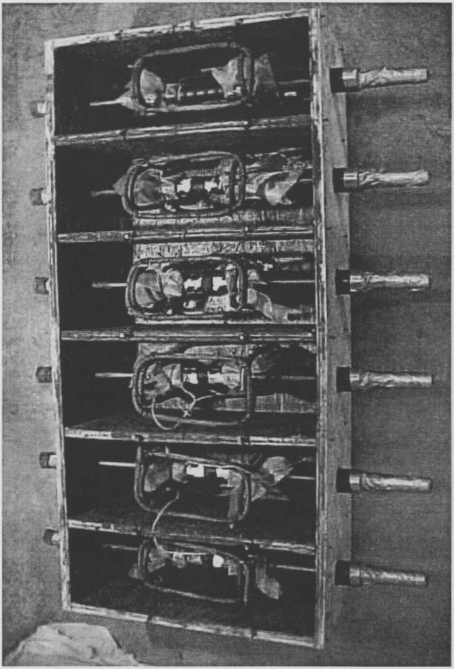

Figure 3.4 – Specimen set up before casting

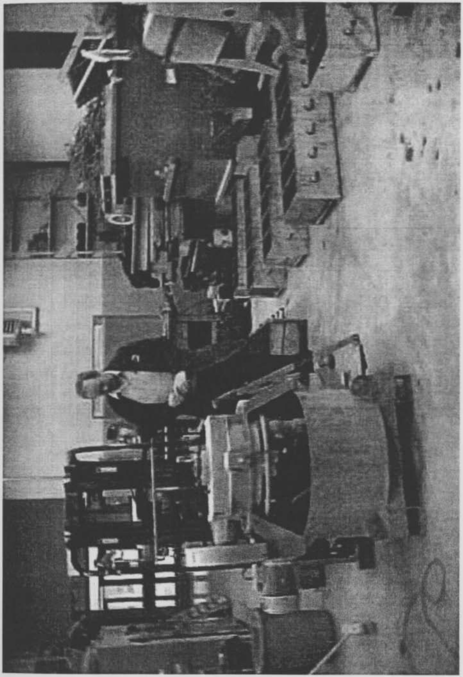

Figure 3.5 – Concrete batching equipment

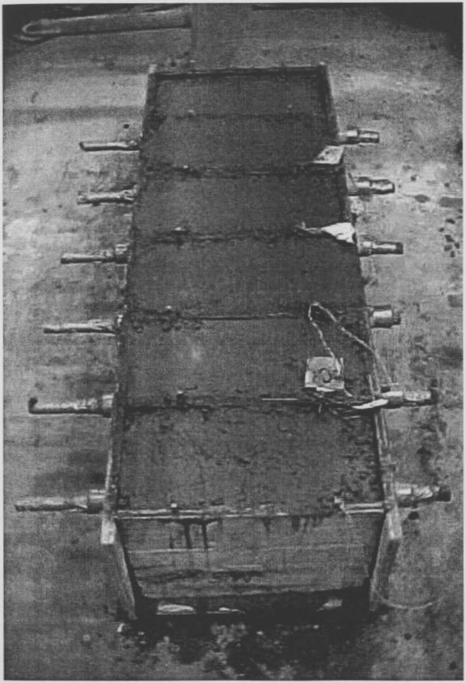

Figure 3.6 – Test specimens in forms after casting

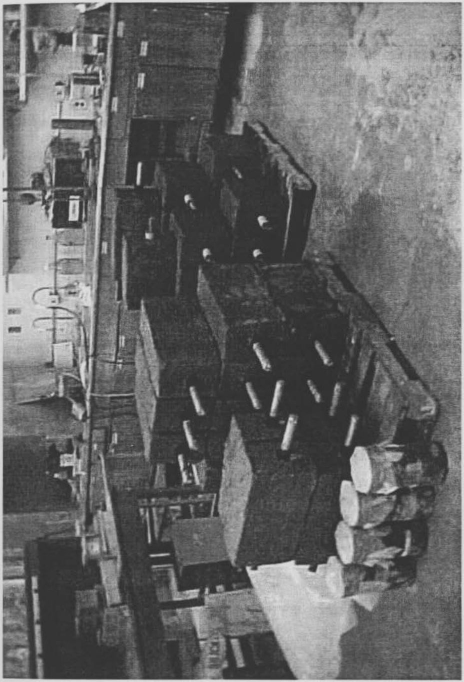

Figure 3.7 – Cured test specimens and cylinders

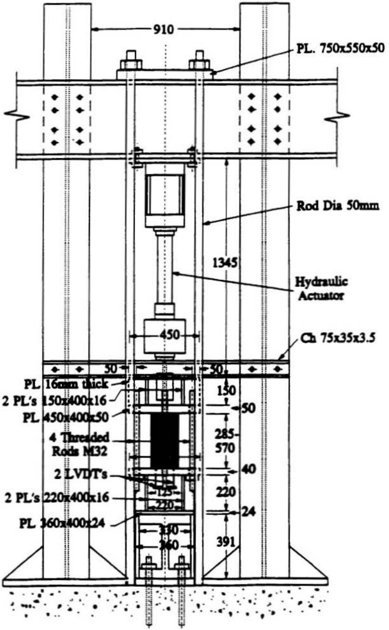

Figure 3.8 – Schematic of test frame set up

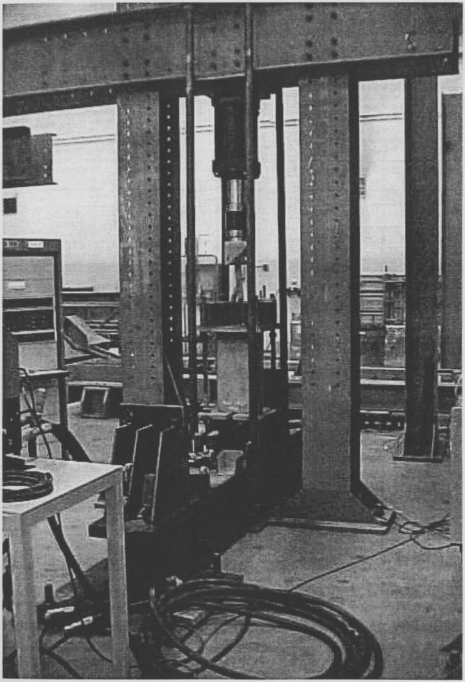

Figure 3.9 – Test frame set up

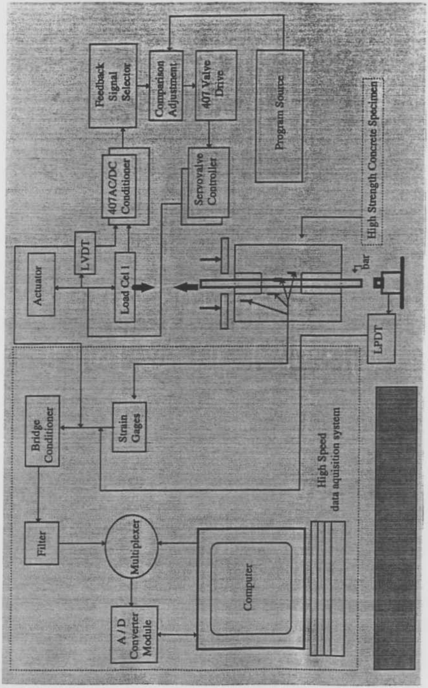

Figure 3.10 – Test instrumentation and close-loop scheme

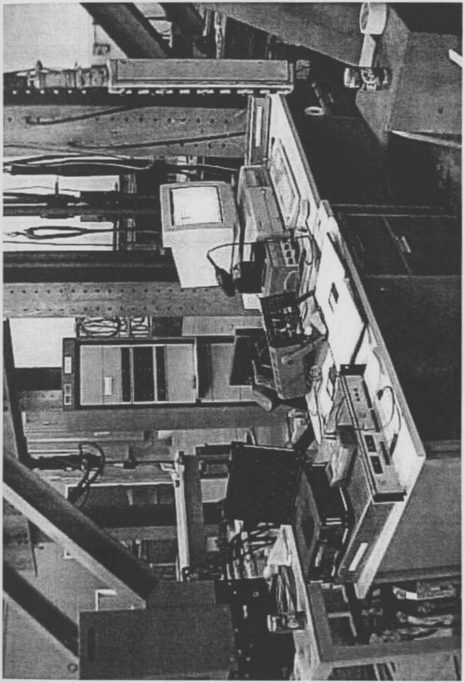

Figure 3.11 – Data Acquisition System

Chapter 4

Reporting of Experimental Results

4.1 Introduction

In efforts to help us understand the bond behavior of deformed reinforcing bars in high strength lightweight (HSLW) a series of experiments were performed on 25 mm and 35 mm deformed reinforcing bars embedded in HSLW concrete. The analytical results and observations of the 70 specimens tested under tension, compression, and various rates of loading as well as cyclic loading are documented throughout this chapter.

4.2 Concrete Properties

The compressive strength of the HSLW concrete used in this experiment was determined as per ASTM C39 (46) to be 83.1 MPa. The test was based on three 150 mm diameter cylinder tests as recorded in Table 4.1. Similarly the average unit weight was measured to be 1810 kg/m^3. Other properties of the concrete such as the modulus of elasticity, splitting tensile strength and modulus of rupture were determined as part of other research work at Memorial University of Newfoundland (MUN) for the same mix design and are described below.

4.2.1 Modulus of Elasticity

The value for the modulus of elasticity, Ec, was determined as per ASTM C469 (47) to be in the range of 26.0 to 27.5 GPa. The modulus of elasticity for HSLW concrete is lower than that of high strength normal weight concrete but higher than normal strength lightweight concrete.

4.2.2 Splitting Tensile Strength

The value of the splitting tensile strength was determined as per ASTM C496 (48). The indirect tension test was used to determine the values of the splitting tensile strength for moist cured HSLW concrete. The values ranged from $5.7\sqrt{f'_c}$ to $7.3\sqrt{f'_c}$ (U.S .Units) for the high strength lightweight mix design.

4.2.3 Modulus of Rupture

The modulus of rupture for mix design was determined using the three point loading test as outlined in ASTM C293 (49). The modulus of rupture was determined to be 5.70 MPa with a ratio of $f_r/\sqrt{f'_c} = 8$ (U.S. Units).

4.3 Test Results and Observations

The instrumentation setup allowed for the measurement of the movement in the free end of the bar by using an external Linear Potential Differential Transducer (LPDT) and the loaded end through the use of a Linear Variable Differential Transducer (LVDT) built into the actuator. The net slip of the bar embedded in concrete was determined by subtracting the bar elongation from the LVDT reading. This slip was plotted against the corresponding bond stress to generate the bond stress vs. slip relationship for the bond behavior for HSLW concrete. A typical stress versus slip curve for 25 mm and 35 mm deformed reinforcing bars embedded in HSLW concrete is shown in Figures 4.1 and 4.2 respectively.

These bond stress versus slip curves show that the initial, nearly linear, ascending portion of the curve represents approximately 7% to 8% of the total slip. This coincides with previous work by Hoff (21) on high strength lightweight concrete, which showed that the slope on the initial ascending portion of the curve is steeper and more linear than that of normal strength concrete. Upon reaching the ultimate load there is a sharp descending portion to approximately 40% of the maximum stress value showing that the concrete is now cracked. The effects of friction and mechanical interlock now engage and the load decreases more gradually for the remaining portion of the curve. The slip continues to increase indicating further crushing of the concrete until the bar can no longer withstand any load. The bond energy, taken to be the area under the curve, is less for high strength lightweight concrete than it is for high strength normal weight concrete. While the initial portions of the stress strain curve are very similar, the decreased shear strength of the lightweight aggregate as compared with normal weight aggregate tends to lower the curve in its final stages thereby decreasing the area under the curve.

4.4 Effect of Loading History

4.4.1 Monotonic in Tension

A total of twenty samples were tested in tension at the standard loading rate of 1.50 mm/min, nine samples using 25 mm bar and eleven samples using 35 mm bar. The maximum applied load and slip in the bar were recorded in the data acquisition system. The maximum load, along with the calculated bond stress and slip are tabulated in Table 4.2 and 4.4 for 25 mm and 35 mm bar respectively. Figure 4.3 shows a comparison of the bond stress versus slip for 25 mm and 35 mm deformed reinforcement under monotonic loading in tension. The graph indicates that the maximum bond stress for a 35 mm bar is lower than the maximum bond stress for a 25 mm bar.

The maximum slip for HSLW concrete can be approximated to be five times the slip corresponding to the maximum load. The measured strain in the 25 mm rebar as recorded from the strain gauge is shown in Figure 4.4. Similarly the measured strain in the 35 mm rebar and the surrounding concrete as recorded from the strain gauges is shown in Figures 4.5 and 4.6. The curve shows an increase in steel strain as the load increases and a decrease in steel strain once the primary crack develops at the descending portion of the curve.

The bond stress versus slip curves for samples of each bar size with and without strain gauges were plotted in Figures 4.7 and 4.8. From these graphs it can be shown that the maximum bond stress for 25 mm bar containing strain gauges is less than for samples without strain gauges. This behavior can be attributed to the removal of bar deformations for approximately 50% of the bond length to accommodate placement of the strain gauges. However, this does not appear to be the case for the 35 mm bar. Placing the strain gauge on the 35 mm bar does not seem to affect the ultimate bond stress. This may be attributed to the size affect of placing the same size strain gauge on a larger bar.

4.4.2 Monotonic in Compression

A total of twenty two specimens were tested in compression at the standard loading rate of 1.50 mm/min. ten samples using 25 mm bar and twelve samples using 35 mm bar. The maximum applied load and slip in the bar were recorded in the data acquisition system. The maximum load, along with the calculated maximum bond stress and slip are tabulated in Table 4.2 and 4.3 for 25 mm and 35 mm bar respectively. Typical stress versus slip curve for the monotonic in compression tests is shown in Figure 4.9 and 4.10 for 25 mm bar and 35 mm bar respectively. Figure 4.11 shows a comparison of the bond stress versus slip curves for 25 mm and 35 mm deformed reinforcement under monotonic loading in compression. The graph indicates that the maximum bond stress for a 25 mm bar is greater than for a 35 mm bar. Figure 4.12 and 4.13 compares the pullout tension test to the push-in compression test for 25 mm and 35 mm. respectively. The slope for the ascending portion of the compression test is higher than that of the tension test indicating that the bond strength is greater in compression than it is in tension for a given bar size. The maximum slip under compression is approximately five times the peak load slip. however when compared to the maximum slip under tension. the maximum slip under compression is approximately 50% the slip under tension. This is due to the cracks opening under tension. thereby resulting in more slip under tension. As a result the area under the curve is less for a bar in compression versus tension indicating that the bond energy is less under compression.

4.5 Effect of Rate of Loading

The rate at which the load is applied was believed to have a significant effect on the maximum bond stress. One sample was tested in tension for each bar size at three loading rates: the standard load rate (1.50 mm/min). fifty times greater than the standard rate (75 mm/min) and ten times less than the standard rate (0.15 mm/min). The test results for the various loading rates are plotted in Figures 4.14 and Figure 4.15 for bar sizes 25 mm and 35 mm. respectively.

For the 25 mm specimens, increasing the rate at which the load was applied does not appear to have a direct affect on the maximum bond stress. The ascending portion of the curves for each loading rate is nearly identical followed by a steep descending portion after the maximum bond stress was achieved.

However, for the 35 mm specimen, there does appear to be a trend that is directly related to the rate of loading. The lower the loading rate, the steeper the ascending portion of the curve, but the maximum bond stress is less. With the limited number of specimens tested in this category, it is difficult to draw any conclusions about the affect of loading rate on the maximum bond capacity.

4.6 Effect of Cyclic Loading

The effect of cyclic loading on the reinforcement in concrete structures is to gradually reduce the bond and to extend the yielding of the bar to within the development length region. This effectively reduces the amount of development length available to develop the yield strength of the bar resulting in pullout of the reinforcement. The lack of research data with regards to the bond behavior of HSLW concrete under cyclic loading often leads to over design and the inefficient use of HSLW concrete.

One sample was tested under cyclic loading for each bar size subjected to the three loading rates of 1.5 mm/min, 75 mm/min and 0.15 mm/min. The loading history was displacement controlled with the first ten cycles set at $\pm$ 3.75 mm and the remaining five cycles set at $\pm$ 7.50 mm. The first level of $\pm$ 3.75 mm was set such that the initial response of the bond strength could be studied without severe damage to the bond strength. while the second level of $\pm$7.50 mm was selected to be close to the maximum slip associated with the maximum bond stress.

The bond behavior under cyclic loading for 25 mm and 35 mm reinforcement is tabulated in Table 4.4 and 4.5 and shown in Figures 4.16 and 4.17 respectively. It can be seen from these plots that the bond is not severely damaged during the first stage of cyclic loading, while increasing the displacement in the second stage of loading causes a rapid deterioration of bond strength.

The bond behavior of HSLW concrete under cyclic loading can be summarized as follows: During the initial loading stages of $\pm$3.75 mm/min, there was not a significant reduction in the maximum bond stress. The first cycle at $\pm$7.50 mm/min saw a significant reduction in the maximum bond stress. This is due to the slip that takes place immediately after the bond is broken and before the ribs of the reinforcement reseat on the concrete. The amount of slip depends on the amount of micro-cracking and inelastic deformation in the vicinity of the ribs. The bond stress continues to deteriorate more gradually due to frictional forces and aggregate interlock. Lastly, the bond stress decreases to a minimum as the effects of the frictional forces and aggregate interlock diminish. As a result of the cracks opening under tension, the slip is greater in tension than compression.

The rate of loading was increased by 50 times to 75 mm/min and decreased by 10 times to 0.15 mm/min for both the 25 mm and 35 mm bars. The data indicated that varying the rate of loading does not have a significant influence on the bond capacity of deformed reinforcement bars subjected to cyclic loading.

Table 4.1 – Compressive Test on HSLW Concrete Cylinders

	Diameter of Cylinder (mm)	Compressive Strength (MPa)
Cylinder 1	150	82.8
Cylinder 2	150	85.8
Cylinder 3	150	80.7
Average	150	83.1

Table 4.2 - Summary of Monotonic Test for 25M Bar

Series	Investigation Parameter	Type of Testing	Specimen File #	Rate of Loading	Peak Load Pmax kN	Bond Length mm	Bond Stress Umax MPa	Normalized Test Results	Peak Load Slip mm
1	Load History	Monotonic in Tension	HSLW 252	1.50	204.87	104	24.87	5.700	11.7
			HSLW 254	1.50	191.58	101	23.95	5.489	11.9
			HSLW 255	1.50	202.91	99	25.88	5.930	12.6
			HSLW 256	1.50	188.54	106	22.46	5.146	14.8
			HSLW 2514	1.50	192.67	100	24.33	5.575	13.3
			HSLW 2515	1.50	193.98	100	24.49	5.613	11.3
			HSLW 2516	1.50	216.41	100	27.32	6.262	10.4
			HSLW 25b1	1.50	146.95	97	19.13	4.383	5.9
			HSLW 25b2	1.50	152.83	100	19.30	4.422	7.2
		Monotonic in Compression	HSLW 2520	1.50	240.82	133	22.86	5.239	9.0
			HSLW 2521	1.50	205.55	111	23.38	5.358	8.0
			HSLW 2522	1.50	236.25	111	26.87	6.158	5.8
			HSLW 2523	1.50	203.81	115	22.38	5.128	5.0
			HSLW 2524	1.50	240.60	133	22.84	5.234	8.0
			HSLW 2525	1.50	284.15	121	29.65	6.795	9.0
			HSLW 2526	1.50	268.25	109	31.07	7.121	6.7
			HSLW 2527	1.50	192.27	126	19.27	4.415	4.7
			HSLW 2528	1.50	255.19	117	27.54	6.311	6.5
			HSLW 25b5	1.50	212.73	109	24.64	5.647	5.0
2	Rate of Loading	Rate of Pullout (Monotonic)	HSLW 258	1.50	205.96	99	26.27	6.019	15.7
			HSLW 2513	1.50	111.46	88	15.99	3.665	3.4
			HSLW 257	75	180.92	104	21.97	5.033	5.9
			HSLW 2510	0.150	188.54	104	22.89	5.245	15.0
			HSLW 2511	0.150	175.26	89	24.86	5.698	9.2
			HSLW 2517	0.150	193.55	105	23.27	5.334	3.4

Table 4.3 - Summary of Monotonic Tests for 35M Bar

Series	Investigation Parameter	Type of Testing	Specimen File #	Rate of Loading	Peak Load Pmax kN	Bond Length mm	Bond Stress Umax MPa	Normalized Test Results	Peak Load Slip mm
3	Load History	Monotonic in Tension	HSLW35a0	1.50	333.98	122	24.40	5.591	7.5
			HSLW35a2	1.50	227.94	87	23.35	5.351	4.6
			HSLW35a3	1.50	313.95	102	27.43	6.287	5.5
			HSLW35a4	1.50	255.60	87	26.19	6.001	5.0
			HSLW35a5	1.50	323.09	104	27.69	6.345	6.7
			HSLW35a7	1.50	281.73	92	27.29	6.254	5.3
			HSLW35a8	1.50	260.83	89	26.12	5.986	4.2
			HSLW35a9	1.50	255.60	92	24.76	5.674	0.1
			HSLW3511	1.50	260.17	99	23.42	5.367	2.8
			HSLW35b0	1.50	223.81	100	19.95	4.571	3.2
			HSLW35b1	1.50	212.27	100	18.92	4.335	3.1
		Monotonic in Compression	HSLW3513	1.50	307.01	102	26.83	6.147	3.8
			HSLW3514	1.50	315.50	105	26.78	6.137	3.5
			HSLW3515	1.50	304.40	108	25.12	5.757	3.2
			HSLW3516	1.50	254.32	108	20.99	4.810	4.9
			HSLW3517	1.50	298.74	97	27.45	6.290	3.8
			HSLW3518	1.50	274.35	94	26.01	5.961	4.2
			HSLW3519	1.50	292.64	96	27.17	6.226	3.2
			HSLW3520	1.50	297.05	100	26.48	6.067	5.5
			HSLW3521	1.50	303.74	104	26.03	5.965	3.5
			HSLW3522	1.50	239.51	98	21.78	4.992	2.5
			HSLW35b2	1.50	221.66	107	18.46	4.231	3.0
			HSLW35b3	1.50	221.88	107	18.48	4.235	19.3
4	Rate of Loading	Rate of Pullout (Monotonic)	HSLW352	1.50	256.03	91	25.08	5.746	1.8
			HSLW355	1.50	270.19	99	24.32	5.574	3.0
			HSLW35a6	75	303.50	90	30.06	6.888	6.3
			HSLW356	0.150	201.38	117	15.34	3.515	0.0

Table 4.4 - Summary of Cyclic Tests for 25M Bar

Series	Investigation Parameter	Type of Testing	Specimen File #	Rate of Loading	Peak Load Pmax kN	Bond Length mm	Bond Stress Umax MPa	Normalized Test Results
2	Rate of Loading	Cyclic	HSLW 2518	1.50	186.80	106	22.25	5.099
			HSLW 2519	1.50	129.97	110	14.92	3.419
			HSLW 2512	75	178.33	99	22.74	5.212

Table 4.5 - Summary of Cyclic Tests for 35M Bar

Series	Investigation Parameter	Type of Testing	Specimen File #	Rate of Loading	Peak Load Pmax kN	Bond Length mm	Bond Stress Umax MPa	Normalized Test Results
2	Rate of Loading	Cyclic	HSLW357	1.50	259.30	104	22.22	5.092
			HSLW358	1.50	196.62	94	18.64	4.272
			HSLW359	75	292.64	102	25.57	5.860
			HSLW3510	0.150	278.89	104	23.90	5.477

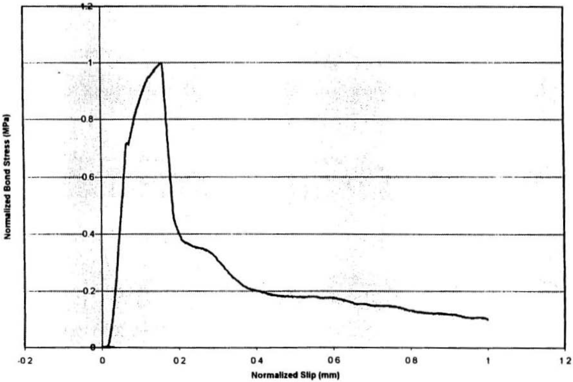

Figure 4.1 Typical Normalized Bond Stress vs Slip for 25mm Bar (Monotonic in Tension)

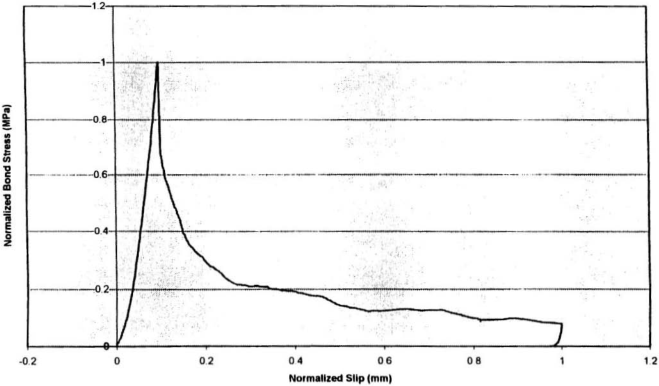

Figure 4.2 - Typical Normalized Bond Stress vs Slip for 35mm Bar

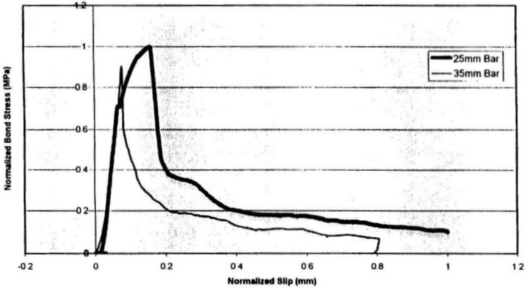

Figure 4.3 Comparison of Normalized Bond Stress vs Slip for 25mm and 35mm Bar (Monotonic in Tension)

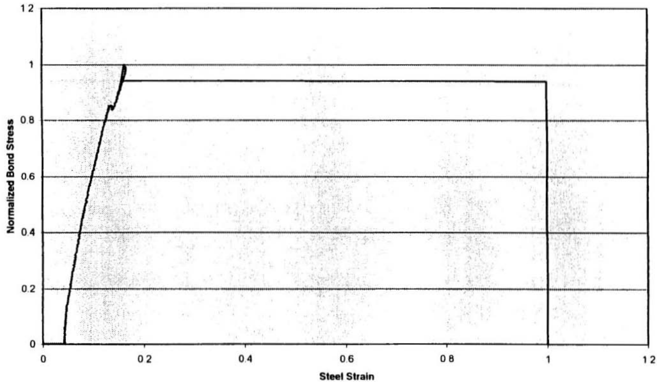

Figure 4.4 - Bond Stress vs Steel Strain for 25mm Bar

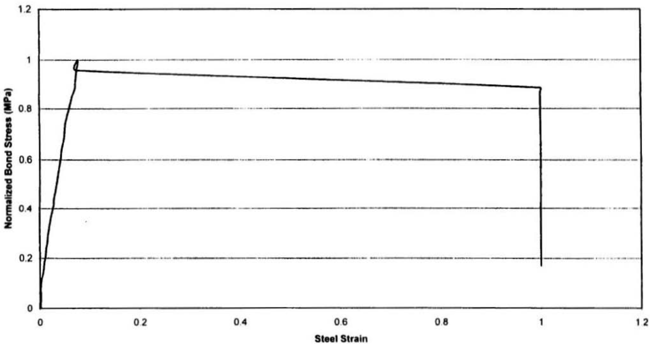

Figure 4.5 - Bond Stress vs Steel Strain for 35mm Bar (Monotonic in Tension)

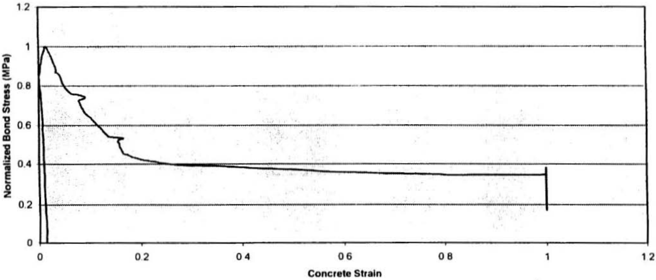

Figure 4.6 - Bond Stress vs Concrete Strain for 35mm Bar (Monotonic in Tension)

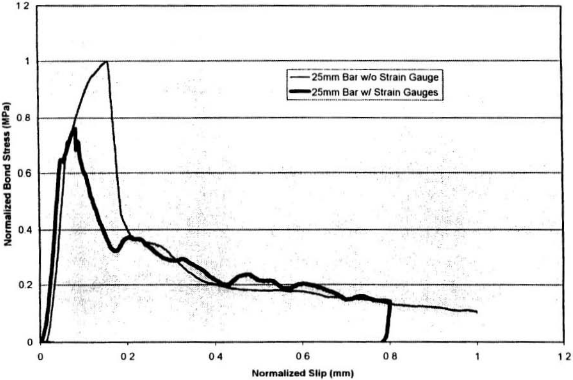

Figure 4.7 - Comparison of Normalized Bond Stress vs Slip for 25mm Bar with Strain Guages (Monotonic in Tension)

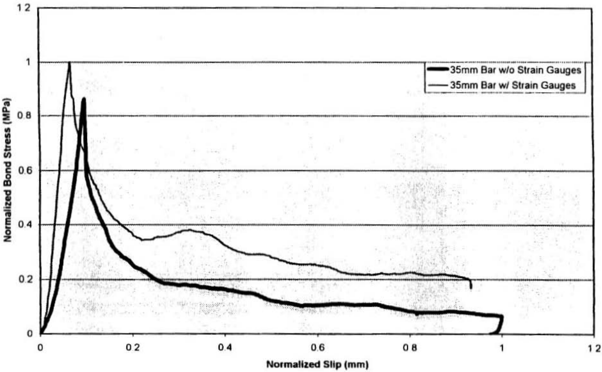

Figure 4.8 - Comparison of Normalized Bond Stress vs Slip for 35mm Bar with Strain Gauges (Monotonic in Tension)

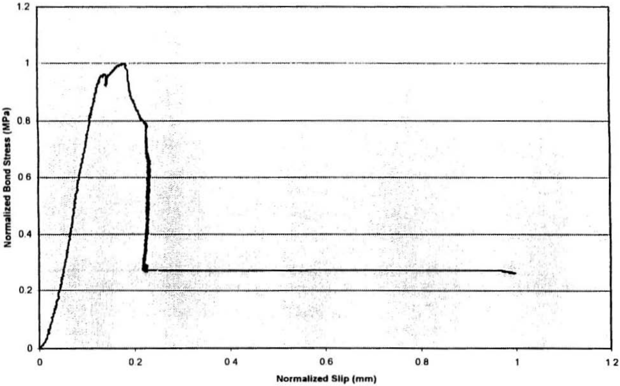

Figure 4.9 - Typical Normalized Bond Stress vs Slip for 25mm Bar (Monotonic in Compression)

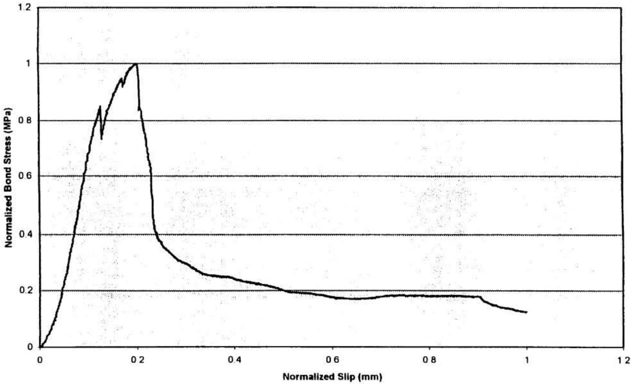

Figure 4.10 - Typical Normalized Bond Stress vs Slip for 35mm Bar (Monotonic in Compression)

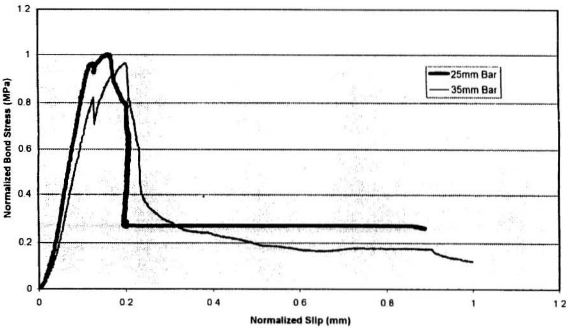

Figure 4.11 - Comparison of Normalized Bond Stress vs Slip for 25mm and 35mm Bar (Monotonic in Compression)

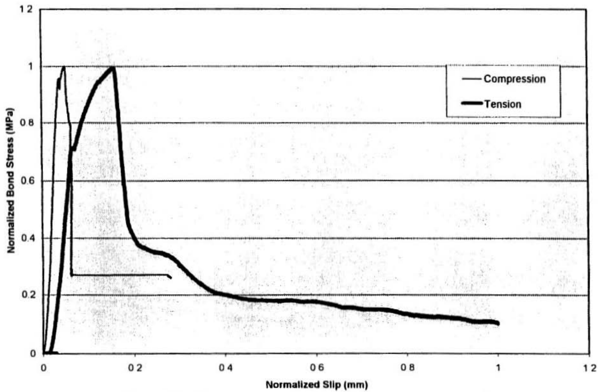

Figure 4.12 - Comparison of Tension and Compression for 25mm Bar

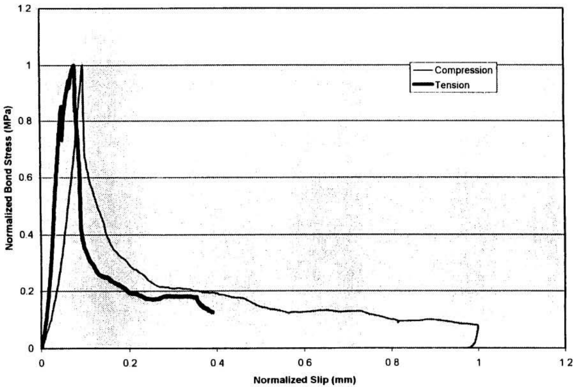

Figure 4.13 - Comparison of Tension and Compression for 35mm Bar

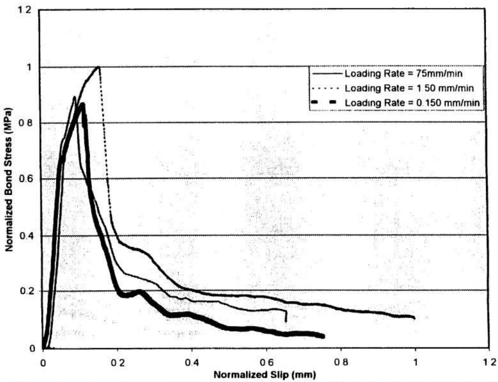

Figure 4.14 - Comparison of Different Loading Rates for 25mm Bar (Monotonic in Tension)

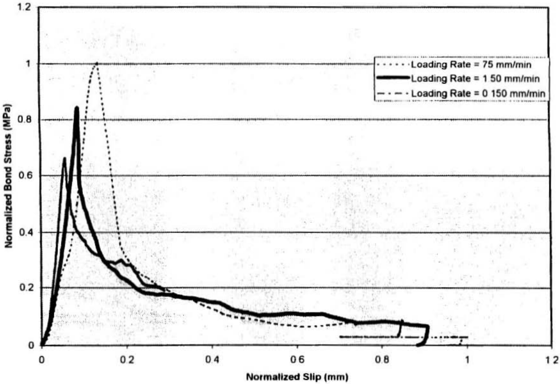

Figure 4.15 - Comparison of Different Loading Rates for 35mm Bar (Monotonic in Tension)

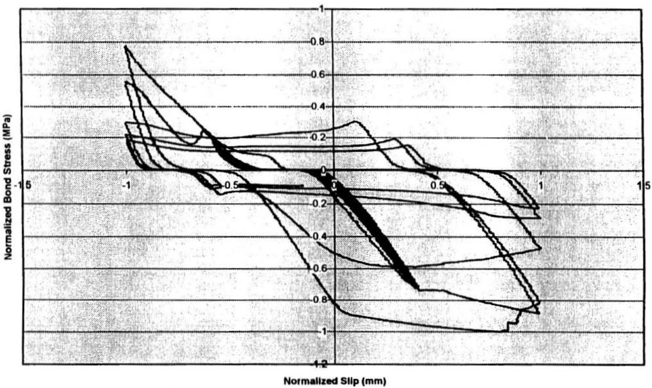

Figure 4.16 - Typical Normalized Bond Stress vs Slip for 25 mm Bar (Cyclic Loading)

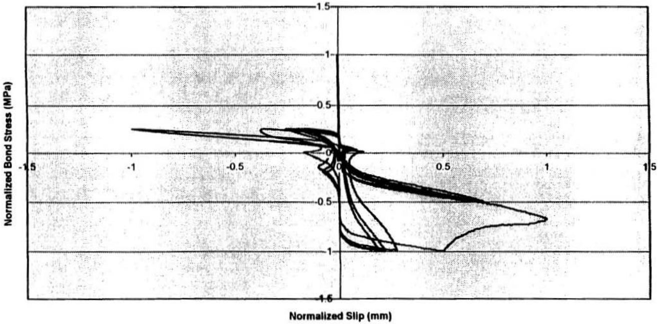

Figure 4.17 - Typical Normalized Bond Stress vs Slip for 35mm Bar (Cyclic Loading)

Chapter 5

Discussion of Results

5.0 Introduction

With the results formally tabulated in Chapter 4 the next step is to analyze the results with respect to how the results compare to those presented in the work of other researchers. The chapter begins with a comparison of the mean values for bond stress and slip from high strength lightweight concrete to that of high strength normal weight concrete for monotonic testing. The next section compares the mean values for bond stress and slip from high strength lightweight concrete to that of high strength normal weight concrete cyclic testing. The third section will concentrate on comparing the test results for HSLW to previous experimental bond research by Esfahani and Rangan (12) and Darwin et al. (12) and concludes with a comparison to the Canadian (2), American (22) and Australian (54) design codes.

The main point of interest in this chapter is to determine if the concrete density modification factor specified in the codes for normal strength lightweight concrete is applicable to high strength lightweight concrete. While the limited number of specimens and parameters tested in this thesis may not be sufficient to permit the proposal of a new bond equation. it is a significant contribution towards the development of future code equations to improve the use of HSLW concrete.

5.1 Comparison of Monotonic Test on HSLW and HSNW Concrete.

The experimental test setup and procedure used in this thesis was modeled after the experimental work performed by Alavi-Fard and Marzouk (37) on high strength normal weight concrete. This was intentional as it was felt that using a similar test setup would allow us to draw direct comparisons between the bond behavior of high strength light weight and high strength normal weight concrete. This section compares the effects of the different loading parameters on the two types of concrete. As was discussed in Chapter 4. the parameters thought to have the most influence on bond behavior were loading history. rate of loading and cyclic loading.

5.1.1 The Effect of Loading History

The mean bond stress along with the standard deviation and 95% confidence level for HSLW and HSNW for specimens tested under monotonic loading in tension and monotonic loading in compression is tabulated in Table 5.1 and Table 5.2 for 25 mm and 35 mm bar respectively. For all cases of 25 mm bar in HSLW concrete, the results indicate that we can be 95% confident that the mean bond stress will be in the range of 5.18 MPa to 5.773 MPa (5.477 $\pm$ 0.296), whereas the mean bond stress for 25 mm bar in

HSNW concrete will be in the range of 5.354 MPa to 6.314 MPa (5.834 $\pm$ 0.480). Similarly, for all cases of 35 mm bar in HSLW concrete, the results indicate that the mean bond stress will be in the range 5.266 MPa to 5.868 MPa (5.567 $\pm$ 0.301), whereas the mean bond stress for 35 mm bar in HSNW concrete will be in the range of 4.182 MPa to 4.832 MPa (4.507 $\pm$ 0.325).

Figures 4.1 through 4.15 show a comparison of the ultimate bond stress versus slip for 25 mm and 35 mm bar in HSLW concrete. Similarly, Figure 5.1 is taken from the work of Alavi- Fard and Marzouk (37) showing ultimate bond stress versus slip for 25 mm and 35 mm bar in HSNW concrete. A comparison with the graphs plotted in Chapter 4 suggest the behavior of HSLW is very similar to HSNW concrete. The results show a relatively steep ascending portion of the curve followed by a steep descending portion after the maximum bond stress level is attained. Albeit, the reduction in bond capacity in the descending portion of the graphs is significantly more for HSLW than for HSNW. Alavi-Fard and Marzouk (37) reported a 30% to 40% decrease in the bond stress during the sharp descending portion of the curve for HSNW, whereas the decrease in bond stress for the descending portion of the HSLW curve is closer to 50% to 60%. This can be attributed to the fact that the lightweight aggregates do not exhibit the same aggregate interlock characteristics of normal weight aggregate. Therefore, the load carrying capacity of HSLW concrete after the maximum load is reached is less than for HSNW.

The trend of the smaller bar size having larger area under bond stress – slip curve is also the case the HSLW concrete. However, the amount of area for the same bar size is much less for HSLW due to the much greater loss of bond strength in the descending portion of the curve. This translates into HSLW concrete being even more brittle and having much less energy absorption capacity than HSNW concrete.

A comparison of the results for compression test show that HSLW concrete exhibit similar behavior to HSNW concrete. The ascending portion of the graph for compression test is steeper than for tension. indicating that the bars have more bond capacity in compression than in tension. This is the same behavior Alavi-Fard and Marzouk (37) reported for HSNW concrete.

5.1.2 The Effect of Rate of Loading

A comparison of the test where the rate of loading was varied indicates that increasing or decreasing the loading rate has minimal affect on the overall bond capacity of the bar. This concurs with the work on HSNW concrete (37) that indicated that changing the rate of loading had no significant affect on the overall bond capacity of the bar. The only noted affect was a slightly steeper gradient on the ascending portion of the curve when the loading rate was decreased. However. this trend was only observed on the 35mm bar and with such limited amount of samples it is difficult to correlate a slower loading rate with a steeper ascending portion of the curve. The ultimate bond stress was not significantly affected by an increase or decrease in loading rate.

5.2 Comparison of Cyclic Test on HSLW and HSNW Concrete

Analysis of the results for the cyclic test conclude that cyclic loading does not have a significant affect on the bond strength provided that the maximum cyclic displacement is less than the peak load slip in a static test. However, once the displacement exceeds this peak load slip than rapid deterioration of the bond capacity exist. This is a similar conclusion than that which was reported in research on HSNW concrete by Alavi-Fard and Marzouk (37).

By examining the bond stress – slip curves one can determine the influence of rate of loading on the bond strength. Figures 4.16 and 4.17 show that the slope of the curve for the initial cycles is constant, however as the amount of slip increases there is a reduction in the slope of the curve. The failure mechanism was initiated by cracking followed by pullout of the bar from the concrete specimen. This is very similar to the failure mode of HSNW concrete (37).

5.3 Comparison of Test Results with other Researchers and Design Codes

This section contains a comparative study of the results of the tests on HSLW concrete with the empirical equations developed by other researchers along with the equations used to calculate bond stress in the Canadian, American and Australian concrete design codes. From this study we will conclude that the concrete density modification factors used in the design equations are too conservative for high strength lightweight concrete.

5.3.1 Development of Different Bond Equations

Alavi-Fard and Marzouk (37) concluded from their work that an equation using the cubic root of the compressive strength of the concrete more accurately represented the bond stress in high strength normal weight concrete. The complete equation as based on the work of Alavi-Fard and Marzouk (37) including modification factors for each of the parameters tested is as follows:

$$\mu = \frac{1.285\,(f'c)^{1/3}}{m} \quad \text{MPa} \qquad (5.1)$$

where: μ = the bond stress in high strength concrete

f'_c = compressive strength of concrete

m = the combined result of the influence m_1, m_2, m_3, m_4, m_5 & m_6

m = 1.0 for purpose of this study

m_1 = effect of load history

m_2 = effect of confinement reinforcement

m_3 = effect of bar size

m_4 = effect of concrete strength

m_5 = effect of bar spacing

m_6 = effect of rate of pullout

Esfahani and Rangan (12) reported that the Australian design code (54) is significantly more conservative than other codes and standards. For this reason the Working Group of the Standards Australia Committee (55) proposed a set of design equations to bring the calculated bond strength on par with other codes. The equations for this section are as follows:

Australian Code AS3600-1994 (54)

$$\mu = \frac{(2a/d_b + 1)\sqrt{f'_c}}{k_1 k_2 \pi} \qquad \textbf{(5.2)}$$

where: μ = bond strength of tensile bars

$2a$ = twice the cover to the bar or the clear spacing between bars which ever is less.

d_b = bar diameter

k_1 = 1.25 for a horizontal bar with more than 300mm of concrete cast below it and 1.0 for all other bars.

k_2 = 1.7 for slabs, 2.2 for longitudinal bars in beams and columns a and 2.4 for all other longitudinal bars.

It is important to note that the beneficial effect of transverse reinforcement is not taken into consideration. Also a similar factor to k_2 is not found in any of the other codes.

Proposals by Working Group BD/2 of Standards Australia Committee (55)

$$\mu = \frac{0.5\sqrt{f'_c}}{k_1 k_2 k_3 k_4} \qquad \textbf{(5.3)}$$

where: μ = ultimate bond stress for bars in tension

f'_c = concrete compressive strength

k_1 = 1.3 for a horizontal bar with more than 300mm of concrete cast below it and 1.0 for all other bars.

k_2 = varies between 0.7 and 1.0 and takes into account concrete cover, bar diameter and clear spacing between the bars.

k_3 = varies between 0.7 and 1.0 and takes into account the effect of transverse reinforcement.

k_4 = varies between 0.7 and 1.0 and takes into account the effect of transverse compressive pressure.

Darwin et al. (56) concluded that $\sqrt{f'_c}$ does not provide an accurate representation of the effect of concrete strength on the bond strength. Using $\sqrt{f'_c}$ results in an underestimation of bond strength for low strength concrete and an overestimation of bond strength for high strength concrete. Replacing $\sqrt{f'_c}$ with $f'^{1/4}_c$ results in more accurate representation of the bond strength for concrete with a compressive strength between 17 and 110 MPa. The revised equation proposed by Darwin et al. (56) is as follows:

$$\mu = \frac{33.75\, f_y}{f_y / (f'_c)^{0.25} - 1900} \qquad \textbf{(5.4)}$$

where: μ = bond stress of tension bars

f_y = yield strength of the reinforcing bar

f'_c = compressive strength of concrete

The American Concrete Institute 318 Building Code (22) calculates the bond strength of deformed reinforcement bars in tension as follows:

$$\mu = \frac{0.651 \sqrt{f'_c}}{\alpha\beta\lambda} \qquad \textbf{(5.5)}$$

where: μ = bond stress in bars in tension

α = bar location factor equal to 1.3 for horizontal reinforcement with more than 300mm of fresh concrete cast below the bar.
α = 1.0 for all other cases.

β = coating factor of 1.5 for epoxy coated reinforcement with less than $3d_b$ concrete cover or clear spacing of less than 6 d_b
β = 1.2 all other epoxy coated reinforcement
β = 1.0 for uncoated reinforcement

λ = 1.3 for structural low density concrete
λ = 1.2 for structural semi-low density concrete
λ = 1.0 for normal density concrete

f'_c = concrete compressive strength

It is important to note that due to safety concerns and lack of test data, the ACI Building Code (22) has an upper limit on the compressive strength of concrete of 70 MPa.

Comparison of the behavior of HSLW concrete can be compared to the behavior of NSLW concrete through using the empirical design code equations of the Canadian Code CSA A23.3 (2) The current code places a limitation on the maximum permissible value of f'_c to be 64MPa, resulting in limited use of concrete with compressive strengths much above this limit. Furthermore, the code also contains coefficients that are expected to provide the factor of safety in the factored bond resistance without the use of factored material strengths used to determine the factored resistances for flexure, shear, axial compression, etc. One such factor used in the empirical equation for development length is a concrete density modification factor. This factor requires that the development length of the reinforcement be increased by a factor of 1.2 or 1.3 for semi low density and low density concrete respectively. These code modification coefficients combined with the upper allowable limit of f'_c has placed severe limitations on the use of HSLW concrete.

In Clause 12.2.3 of CSA A23.3 (2) the minimum development length l_d for deformed bars in tension can be determined from $l_d = 0.45\ (k_1\ k_2\ k_3\ k_4) \cdot f_y \cdot d_b/(f'c)^{0.5}$

Where: k_1 = bar location factor equal to 1.3 for horizontal reinforcement with more than 300mm of fresh concrete cast below the bar.
k_1 = 1.0 for all other cases.

k_2 = coating factor of 1.5 for epoxy coated reinforcement with less than $3d_b$ concrete cover or clear spacing of less than 6 d_b
k_2 = all other epoxy coated reinforcement
k_2 = for uncoated reinforcement

k_3 = 1.3 for structural low density concrete
k_3 = 1.2 for structural semi-low density concrete
k_3 = 1.0 for normal density concrete

k_4 = 0.8 for 20mm and smaller bars
k_4 = 1.0 for 25mm and larger bars

So for the case of our experiment the bond stress μ can be determined as:

$$\mu = F/A$$

where, $F = fy \times A_b$

and, $A = \pi \times d_b \times l_d$

therefore, $\mu = (0.556 \times \sqrt{f'_c})/K$ **(5.6)**

where, $K = k_1 \times k_2 \times k_3 \times k_4$

Table 5.5 and 5.6 compares the bond stress from the test results to the calculated values using the six different formulae discussed in this section.

5.3.2 Discussion of Comparative Study

The bond strength of concrete is proportional to the tensile strength. However, the industry standard is to determine the bond behavior based on the compressive strength, mainly due to the ease of testing in compression using the standard cylinder or cube test. As a result a bond expression based on the square root of the compressive strength is commonly used in design codes around the world to represent the relationship between the tensile strength and the compressive strength of concrete. However, for high strength concrete the use of the square root of the compressive strength can lead to an overestimation of the tensile strength and subsequently an overestimation of the bond capacity. Taking the bond strength of high strength concrete to be proportional to the cubic root of the compressive strength more accurately represents the tensile strength. From Table 5.6 and 5.8 the expression used by Alavi-Fard and Marzouk (37) best fits the experimental data for high strength lightweight concrete. However, given that the test consisted of testing one bar using this expression may not produce a large enough factor of safety. It was discuss in ACI 318 (22) that for multiple bars being developed in one area a plane of failure may develop at lower stress levels than that resisted by the bond of a single bar. As a result a more practical solution for HSLW concrete is to use a concrete density modification factor k_3 equal to 1.1 instead of 1.3. From Table 5.6 and 5.8 it can be seen that k_3 equal to 1.1 results in a ratio of test result to code calculation of 1.20.

Table 5.1 - Comparison of average bond stress for 25M bar in HSLW and HSNW concrete.

			HSLW					HSNW				
Series	Investigation Parameter	Type of Testing	Specimen File #	Normalized Bond Stress MPa	Mean Bond Stress MPa	S.D. MPa	95% Confidence Level	Specimen File #	Normalized Bond Stress MPa	Mean Bond Stress MPa	S.D. Mpa	95% Confidence Level
1	Load History	Monotonic in Tension	HSLW 252	5.700	5.391	0.601	0.393	1HNM-19-1	6.958	6.083	0.578	0.564
			HSLW 254	5.489				1HNM-19-1A	6.194			
			HSLW 255	5.930				1HNM-19-1B	5.415			
			HSLW 256	5.146				1HNM-19-1C	5.763			
			HSLW 2514	5.575								
			HSLW 2515	5.613								
			HSLW 2516	6.262								
			HSLW 25b1	4.383								
			HSLW 25b2	4.422								
		Monotonic in Compression	HSLW 2520	5.239	5.741	0.794	0.492	1HNM-19-2	5.356	5.961	0.674	0.660
			HSLW 2521	5.358				1HNM-19-2A	5.498			
			HSLW 2522	6.158				1HNM-19-2B	5.919			
			HSLW 2523	5.128				1HNM-19-2C	7.072			
			HSLW 2524	5.234								
			HSLW 2525	6.795								
			HSLW 2526	7.121								
			HSLW 2527	4.415								
			HSLW 2528	6.311								
			HSLW 25b5	5.647								
2	Rate of Loading	Rate of Pullout (Monotonic)	HSLW 258	6.019	5.166	0.743	0.595	6HNM-6-2	5.919	5.081	0.838	0.948
			HSLW 2513	3.665				6HNM-6-3	4.243			
			HSLW 257	5.033								
			HSLW 2510	5.245								
			HSLW 2511	5.698								
			HSLW 2517	5.334								
			All test in Group 1& 2		5.477	0.754	0.296	All test in Group 1& 2		5.834	0.774	0.480

Table 5.2 - Comparison of average bond stress for 35M bar in HSLW and HSNW concrete

Series	Investigation Parameter	Type of Testing	HSLW Specimen File #	HSLW Normalized Bond Stress MPa	HSLW Mean Bond Stress MPa	HSLW S.D. MPa	HSLW 95% Confidence Level	HSNW Specimen File #	HSNW Normalized Bond Stress MPa	HSNW Mean Bond Stress MPa	HSNW S.D. MPa	HSNW 95% Confidence Level
3	Load History	Monotonic in Tension	HSLW35a0	5.591	5.615	0.642	0.380	1HNM-19-1	5.458	4.440	0.708	0.694
			HSLW35a2	5.351				1HNM-19-1A	4.499			
			HSLW35a3	6.287				1HNM-19-1B	3.462			
			HSLW35a4	6.001				1HNM-19-1C	4.341			
			HSLW35a5	6.345								
			HSLW35a7	6.254								
			HSLW35a8	5.986								
			HSLW35a9	5.674								
			HSLW3511	5.367								
			HSLW35b0	4.571								
			HSLW35b1	4.335								
		Monotonic in Compression	HSLW3513	6.147	5.568	0.746	0.422	1HNM-19-2A	4.511	4.278	0.265	0.299
			HSLW3514	6.137				1HNM-19-2B	4.415			
			HSLW3515	5.757				1HNM-19-2C	3.908			
			HSLW3516	4.810								
			HSLW3517	6.290								
			HSLW3518	5.961								
			HSLW3519	6.226								
			HSLW3520	6.087								
			HSLW3521	5.965								
			HSLW3522	4.992								
			HSLW35b2	4.231								
			HSLW35b3	4.235								
4	Rate of Loading	Rate of Pullout (Monotonic)	HSLW352	5.746	5.431	1.216	1.191	6HNM-6-1	4.711	4.825	0.131	0.148
			HSLW355	5.574				6HNM-6-2	5.008			
			HSLW35a6	6.888				6HNM-6-3	4.756			
			HSLW356	3.515								
			All test in Group 3 & 4		5.567	0.799	0.301	All test in Group 3 & 4		4.507	0.524	0.325

Table 6.3 - Comparison of average slip values for 25M bar in HSLW and HSNW concrete.

			HSLW					HSNW				
Series	Investigation Parameter	Type of Testing	Specimen File #	Slip at Peak Stress mm	Mean Slip mm	S.D. mm	95% Confidence Level	Specimen File #	Slip at Peak Stress mm	Mean Slip mm	S.D. mm	95% Confidence Level
1	Load History	Monotonic in Tension	HSLW 252	11.70	11.01	2.68	1.75	1HNM-19-1	8.19	7.22	1.82	1.78
			HSLW 254	11.90				1HNM-19-1A	9.67			
			HSLW 255	12.60				1HNM-19-1B	5.92			
			HSLW 256	14.80				1HNM-19-1C	5.08			
			HSLW 2514	13.30								
			HSLW 2515	11.30								
			HSLW 2516	10.40								
			HSLW 25b1	5.90								
			HSLW 25b2	7.20								
		Monotonic in Compression	HSLW 2520	9.00	6.77	1.57	0.97	1HNM-19-2	5.52	4.95	0.64	0.63
			HSLW 2521	8.00				1HNM-19-2A	5.12			
			HSLW 2522	5.80				1HNM-19-2B	3.86			
			HSLW 2523	5.00				1HNM-19-2C	5.29			
			HSLW 2524	8.00								
			HSLW 2525	9.00								
			HSLW 2526	6.70								
			HSLW 2527	4.70								
			HSLW 2528	6.50								
			HSLW 25b5	5.00								
2	Rate of Loading	Rate of Pullout (Monotonic)	HSLW 258	15.70	8.77	5.05	1.22	6HNM-6-2	5.29	6.240	0.950	1.075
			HSLW 2513	3.40				6HNM-6-3	7.19			
			HSLW 257	5.90								
			HSLW 2510	15.00								
			HSLW 2511	9.20								
			HSLW 2517	3.40								
			All test in Group 1 & 2		8.78	3.62	1.42	All test in Group 1 & 2		6.11	1.64	1.02

Table 5.4 - Comparison of average slip values for 35M bar in HSLW and HSNW concrete

Series	Investigation Parameter	Type of Testing	HSLW					HSNW				
			Specimen File #	Slip at Peak Stress mm	Mean Slip mm	S.D. mm	95% Confidence Level	Specimen File #	Slip at Peak Stress mm	Mean Slip mm	S.D. mm	95% Confidence Level
3	Load History	Monotonic in Tension	HSLW35a0	7.50	4.36	1.94	1.15	1HNM-19-1	7.71	6.39	0.89	0.88
			HSLW35a2	4.60				1HNM-19-1A	6.53			
			HSLW35a3	5.50				1HNM-19-1B	5.24			
			HSLW35a4	5.00				1HNM-19-1C	6.06			
			HSLW35a5	6.70								
			HSLW35a7	5.30								
			HSLW35a8	4.20								
			HSLW35a9	0.10								
			HSLW3511	2.80								
			HSLW35b0	3.20								
			HSLW35b1	3.10								
		Monotonic in Compression	HSLW3513	3.80	5.03	4.37	2.47	1HNM-19-2A	6.71	6.37	0.46	0.52
			HSLW3514	3.50				1HNM-19-2B	6.69			
			HSLW3515	3.20				1HNM-19-2C	5.72			
			HSLW3516	4.90								
			HSLW3517	3.80								
			HSLW3518	4.20								
			HSLW3519	3.20								
			HSLW3520	5.50								
			HSLW3521	3.50								
			HSLW3522	2.50								
			HSLW35b2	3.00								
			HSLW35b3	19.30								
4	Rate of Loading	Rate of Pullout (Monotonic)	HSLW352	1.80	2.78	2.30	2.25	6HNM-6-1	6.85	6.60	0.44	0.50
			HSLW355	3.00				6HNM-6-2	6.96			
			HSLW35a6	6.30				6HNM-6-3	5.98			
			HSLW356	0.00								
			All test in Group 3 & 4		4.43	3.37	1.27	All test in Group 3 & 4		6.45	0.67	0.42

Table 6.6 - Comparison of Normalized Test Results for 25mm bar with Various Bond Equations.

Series	Investigation Parameter	Type of Testing	Specimen File #	Normalized Bond Stress**	Various Bond Expressions (A & M) $m = 1$	(E & R)	(AUS)	(DAR)	(ACI) $k_1 = 1.3$	(CSA) $k_1 = 1.3$	(CSA) $k_1 = 1.1$
1	Loading History	Monotonic in Tension	HSLW 252	5.70	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 254	5.49	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 255	5.94	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 256	5.15	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 2514	5.58	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 2515	5.62	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 2516	6.27	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 25b1	4.39	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 25b2	4.43	5.61	4.56	6.04	3.60	2.91	3.90	4.61
		Monotonic in Compression	HSLW 2520	5.24	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 2521	5.36	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 2522	6.16	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 2523	5.13	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 2524	5.24	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 2525	6.60	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 2526	7.13	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 2527	4.42	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 2528	6.32	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 25b5	5.65	5.61	4.56	6.04	3.60	2.91	3.90	4.61
2	Rate of Loading	Rate of Pullout (Monotonic)	HSLW 258	6.03	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 2513	3.67	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 257	5.04	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 2510	5.25	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 2511	5.70	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			HSLW 2517	5.34	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			Mean	5.48	5.61	4.56	6.04	3.60	2.91	3.90	4.61
			SD	0.75							

**Maximum bond stress normalized to: $f'c^{1/2}$

(A & M) - Alavi Fard and Marzouk (30)
(E & R) - Esfahani and Rangan (12)
(AUS) - Australian Standard for Concrete Structures (38)
(DAR) - Darwin et al (13)
(ACI) - ACI 318 Building Code (41)
(CSA) - CSA A23.3 Design of Concrete Structures (2)

Table 5.8 - Comparison of Normalized Test Results for 25mm bar with Various Bond Equations.

Series	Investigation Parameter	Type of Testing	Specimen File #	Normalized Bond Stress**	Various Bond Expressions						
					Test (A & M) m = 1	Test (E & R)	Test (AUS)	Test (DAR)	Test (ACI) $k_1 = 1.3$	Test (CSA) $k_1 = 1.3$	Test (CSA) $k_1 = 1.1$
1	Loading History	Monotonic in Tension	HSLW 252	5.70	1.02	1.25	0.94	1.58	1.96	1.46	1.24
			HSLW 254	5.49	0.98	1.20	0.91	1.53	1.89	1.41	1.19
			HSLW 255	5.94	1.06	1.30	0.98	1.65	2.04	1.52	1.29
			HSLW 256	5.15	0.92	1.13	0.85	1.43	1.77	1.32	1.12
			HSLW 2514	5.58	0.99	1.22	0.92	1.55	1.92	1.43	1.21
			HSLW 2515	5.62	1.00	1.23	0.93	1.56	1.93	1.44	1.22
			HSLW 2516	6.27	1.12	1.37	1.04	1.74	2.15	1.61	1.36
			HSLW 25b1	4.39	0.78	0.96	0.73	1.22	1.51	1.12	0.95
			HSLW 25b2	4.43	0.79	0.97	0.73	1.23	1.52	1.13	0.96
		Monotonic in Compression	HSLW 2520	5.24	0.93	1.15	0.87	1.46	1.80	1.34	1.14
			HSLW 2521	5.36	0.96	1.18	0.89	1.49	1.84	1.37	1.16
			HSLW 2522	6.16	1.10	1.35	1.02	1.71	2.12	1.58	1.34
			HSLW 2523	5.13	0.91	1.13	0.85	1.43	1.76	1.32	1.11
			HSLW 2524	5.24	0.93	1.15	0.87	1.46	1.80	1.34	1.14
			HSLW 2525	6.80	1.21	1.49	1.13	1.89	2.34	1.74	1.48
			HSLW 2526	7.13	1.27	1.56	1.18	1.98	2.45	1.83	1.55
			HSLW 2527	4.42	0.79	0.97	0.73	1.23	1.52	1.13	0.96
			HSLW 2528	6.32	1.13	1.39	1.05	1.76	2.17	1.62	1.37
			HSLW 25b5	5.65	1.01	1.24	0.94	1.57	1.94	1.45	1.23
2	Rate of Loading	Rate of Pullout (Monotonic)	HSLW 258	6.03	1.07	1.32	1.00	1.67	2.07	1.54	1.31
			HSLW 2513	3.67	0.65	0.80	0.61	1.02	1.26	0.94	0.80
			HSLW 257	5.04	0.90	1.11	0.83	1.40	1.73	1.29	1.09
			HSLW 2510	5.25	0.94	1.15	0.87	1.46	1.80	1.35	1.14
			HSLW 2511	5.70	1.02	1.25	0.94	1.58	1.96	1.46	1.24
			HSLW 2517	5.34	0.95	1.17	0.88	1.48	1.83	1.37	1.16
			Mean	5.48	0.98	1.20	0.91	1.52	1.88	1.41	1.19
			SD	0.75	0.13	0.17	0.12	0.21	0.26	0.19	0.16

**Maximum bond stress normalized to: $f'c^{1/2}$

(A & M) - Alavi Fard and Marzouk (30)
(E & R) - Esfahani and Rangan (12)
(AUS) - Australian Standard for Concrete Structures (38)
(DAR) - Darwin et al (13)
(ACI) - ACI 318 Building Code (41)
(CSA) - CSA A23.3 Design of Concrete Structures (2)

Table 5.7 - Comparison of Normalized Test Results for 35mm bar with Various Bond Equations.

Series	Investigation Parameter	Type of Testing	Specimen File #	Normalized Bond Stress**	Various Bond Expressions (A & M) $m = 1$	(E & R)	(AUS)	(DAR)	(ACI) $k_1 = 1.3$	(CSA) $k_1 = 1.3$	(CSA) $k_1 = 1.1$
3	Loading History	Monotonic in Tension	HSLW 35a0	5.59	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 35a2	5.35	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 35a3	6.29	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 35a4	6.00	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 35a5	6.35	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 35a7	6.25	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 35a8	5.99	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 35a9	5.67	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 3511	5.37	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 35b0	4.57	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 35b1	4.34	5.61	5.36	4.66	3.60	2.91	3.90	4.61
		Monotonic in Compression	HSLW 3513	6.15	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 3514	6.14	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 3515	5.76	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 3516	4.81	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 3517	6.29	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 3518	5.96	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 3519	6.23	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 3520	6.07	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 3521	5.97	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 3522	4.99	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 35b2	4.23	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 35b3	4.24	5.61	5.36	4.66	3.60	2.91	3.90	4.61
4	Rate of Loading	Rate of Pullout (Monotonic)	HSLW 352	5.75	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 355	5.57	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 35a6	6.89	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			HSLW 356	3.52	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			Mean	5.67	5.61	5.36	4.66	3.60	2.91	3.90	4.61
			SD.	0.61							

**Maximum bond stress normalized to: $f'c^{1/2}$

(A & M) - Alavi Fard and Marzouk (30)
(E & R) - Esfahani and Rangan (12)
(AUS) - Australian Standard for Concrete Structures (38)
(DAR) - Darwin et al (13)
(ACI) - ACI 318 Building Code (41)
(CSA) - CSA A23.3 Design of Concrete Structures (2)

Table 5.8 - Comparison of Normalized Test Results for 35mm bar with Various Bond Equations.

Series	Investigation Parameter	Type of Testing	Specimen File #	Normalized Bond Stress**	Various Bond Expressions: Test (A & M) m = 1	Test (E & R)	Test (AUS)	Test (DAR)	Test (ACI) $k_2 = 1.3$	Test (CSA) $k_2 = 1.3$	Test (CSA) $k_2 = 1.1$
3	Loading History	Monotonic in Tension	HSLW 35a0	5.59	1.00	1.04	1.20	1.55	1.92	1.43	1.21
			HSLW 35a2	5.35	0.95	1.00	1.15	1.49	1.84	1.37	1.16
			HSLW 35a3	6.29	1.12	1.17	1.35	1.75	2.16	1.61	1.36
			HSLW 35a4	6.00	1.07	1.12	1.29	1.67	2.06	1.54	1.30
			HSLW 35a5	6.35	1.13	1.18	1.36	1.76	2.18	1.63	1.38
			HSLW 35a7	6.25	1.11	1.17	1.34	1.74	2.15	1.60	1.36
			HSLW 35a8	5.99	1.07	1.12	1.29	1.66	2.06	1.54	1.30
			HSLW 35a9	5.67	1.01	1.06	1.22	1.58	1.95	1.45	1.23
			HSLW 3511	5.37	0.96	1.00	1.15	1.49	1.85	1.38	1.16
			HSLW 35b0	4.57	0.81	0.85	0.98	1.27	1.57	1.17	0.99
			HSLW 35b1	4.34	0.77	0.81	0.93	1.21	1.49	1.11	0.94
		Monotonic in Compression	HSLW 3513	6.15	1.10	1.15	1.32	1.71	2.11	1.58	1.33
			HSLW 3514	6.14	1.09	1.15	1.32	1.71	2.11	1.57	1.33
			HSLW 3515	5.76	1.03	1.07	1.24	1.60	1.98	1.48	1.25
			HSLW 3516	4.81	0.86	0.90	1.03	1.34	1.65	1.23	1.04
			HSLW 3517	6.29	1.12	1.17	1.35	1.75	2.16	1.61	1.36
			HSLW 3518	5.96	1.06	1.11	1.28	1.66	2.05	1.53	1.29
			HSLW 3519	6.23	1.11	1.16	1.34	1.73	2.14	1.60	1.35
			HSLW 3520	6.07	1.08	1.13	1.30	1.69	2.09	1.56	1.32
			HSLW 3521	5.97	1.06	1.11	1.28	1.66	2.05	1.53	1.30
			HSLW 3522	4.99	0.89	0.93	1.07	1.39	1.71	1.28	1.08
			HSLW 35b2	4.23	0.75	0.79	0.91	1.18	1.45	1.08	0.92
			HSLW 35b3	4.24	0.76	0.79	0.91	1.18	1.46	1.09	0.92
4	Rate of Loading	Rate of Pullout (Monotonic)	HSLW 352	5.75	1.02	1.07	1.23	1.60	1.98	1.47	1.25
			HSLW 355	5.57	0.99	1.04	1.20	1.55	1.91	1.43	1.21
			HSLW 35a6	6.89	1.23	1.29	1.48	1.91	2.37	1.77	1.49
			HSLW 356	3.52	0.63	0.66	0.76	0.98	1.21	0.90	0.76
			Mean	5.67	0.99	1.04	1.19	1.55	1.91	1.43	1.21
			SD	0.90	0.14	0.16	0.17	0.22	0.27	0.20	0.17

**Maximum bond stress normalized to: $f'c^{1/2}$

(A & M) - Alavi Fard and Marzouk (30)
(E & R) - Esfahani and Rangan (12)
(AUS) - Australian Standard for Concrete Structures (38)
(DAR) - Darwin et al (13)
(ACI) - ACI 318 Building Code (41)
(CSA) - CSA A23.3 Design of Concrete Structures (2)

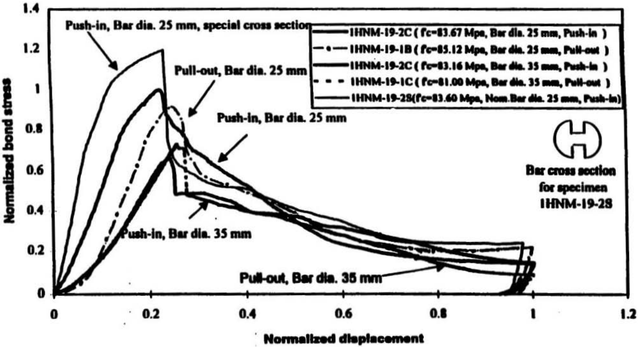

Figure 5.1 – Comparison of normalized bond stress – displacement response for typical specimens due to pull-out and push-in test with an embedded bar diameter of 35 mm and 25 mm.

Chapter 6

> *" If a builder has built a house for a man and has not made his work sound, and the house which he has built has fallen down and so caused the death of the householder, that builder shall be put to death."*
> *Code of Hammurabi, c. 2040 B.C.*

Conclusions and Recommendations

6.1 Conclusions

Researchers are constantly researching and developing new materials and improved methods for construction. This process usually involves extensive testing and a thorough investigation into how these new materials perform under a variety of circumstances to which they are subject to in the real world. The various design code groups throughout the world are given the task of formalizing the use of these materials through the implementation of material specifications and performance criteria. Often when there is limited research data available conservative restrictions are placed on the use of these materials in the interest of public safety. Such is the case with high strength lightweight concrete, and hence the purpose for this research.

It was believed by the author that the restrictions placed on normal strength lightweight concrete through the use of a concrete density modification factor is not justified on high strength lightweight concrete. The experimental investigation consisted of constructing 36 specimens for each for 25mm and 35mm bar. To facilitate a direct comparison with the previous work by Alavi-Fard and Marzouk (37) on high strength normal weight concrete the same specimen size, casting position and test setup was used. The test program consisted of both monotonic and cyclic testing for each bar size. In addition under the monotonic test the effect of loading in tension was compared to compression, as well as varying the rate at which the load was applied. Similarly, the rate of loading was varied in the cyclic test to investigate the effect of increasing the rate of loading by 50 times the nominal rate and decreasing the rate by 10 times the nominal rate.

The test results revealed that high strength lightweight (HSLW) concrete behaves very similar to high strength normal weight concrete (HSNW). The maximum bond stress for HSLW is greater than that of normal strength lightweight concrete. However, the behavior of HSLW concrete is more brittle than normal strength lightweight. As was the case for HSNW concrete the maximum slip value associated with bond failure was approximately five times the slip value corresponding to the maximum bond stress. The shape of the stress – displacement curve for HSLW is very similar to HSNW. The curves begin with a sharp nearly linear ascending portion of the curve, followed by a steep descending portion indicating very brittle behavior. However, the decrease in bond stress for the descending portion of the HSLW curve is 50% to 60% as compared with 30% to 40% for HSNW. This is attributed to lightweight aggregate not having the same aggregate interlock characteristics of normal weight aggregate.

The cyclic test on HSLW concrete conclude that cyclic loading does not have a significant affect on the bond strength provided that the maximum cyclic displacement is less that the peak load displacement in the static test. The stress displacement curve shows that the slope of the initial cycles is constant; however there is a reduction in the slope once the slip associated with maximum stress is exceeded. This is indicative of loss of structural capacity and complete failure by means of pullout of the bar.

The results of the HSLW testing were also compared to the bond expressions developed by other researchers as well as the Australian, American and Canadian concrete design codes. The Canadian concrete design code requires that for normal strength lightweight concrete the basic development length must be multiplied by a concrete density modification factor of 1.3. A comparison of the test results with the code equation concludes that the 30% increase in the development length is too conservative for HSLW concrete, and that if $\sqrt{f'_c}$ is to be used then the concrete density modification factor should be 1.1 for high strength lightweight concrete. It was also determined that the cubic root of the concrete compressive strength better describes the bond behavior of HSLW concrete than the square root of the compressive strength which is currently used in the design codes. The expression developed by Alavi-Fard and Marzouk (37) best represented the experimental data on high strength lightweight concrete.

6.2 Recommendations

It is important to realize that the experimental investigation carried out as part of this thesis was limited to 36 specimens for each bar size and to one type of specimen. The author recommends that more samples and different specimen configurations such as multiple bars, beam end specimens and full scale structural members be tested. This would enable a more comprehensive statistical analysis on the results and the development of a new equation for calculating the bond strength of high strength lightweight concrete.

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