PERFORMANCE OF HIGH STRENGTH LIGHTWEIGHT AGGREGATE CONCRETE IN A SIMULATED MARINE ENVIRONMENT

By

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MARINE ENVIRONMENT

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CHAPTER I

INTRODUCTION

1.1 General Background

During the past century, concrete has been used to construct harbor facilities, piles, barges and ships, tanks, and other structures subjected to a marine environment. Because such applications involve exposure to seawater at a negligible hydrostatic pressure, structural design has been based on the material properties of concrete established by ordinary tests conducted in air. However, as offshore oil and gas production continue to move into deeper water locations, concrete structures are being subjected to greater hydrostatic pressure. Gravity-base concrete structures have been installed where the water depth is 200 m and structures are being planned for sites with much greater depth.

During the design of the early gravity-base structures, designers assumed that the hydrostatic pressure acting on concrete would probably provide a beneficial confinement to the concrete. When concrete is initially subjected to a hydrostatic environment, the water pressure in internal voids is less than the surrounding water. This pressure differential induces stresses in the granular structure of concrete. The pressure differential also causes water to move into the concrete at a rate related to the permeability of the concrete. Voids initially filled with air tend to restrict the development of pore pressure in concrete. However, with time water can enter these voids, compressing and absorbing the air. If the voids become filled with water, the concrete is saturated, and the pressure in pores can be in equilibrium with the pressure in the surrounding water. If concrete reaches this saturated state, there is no confining action resulting from external hydrostatic pressure.

Investigations conducted since 1970 have indicated that normal density (ND) concrete exhibits a strength reduction when tested in a water environment. However, very few investigations have been performed on lightweight aggregate (LWA) concrete which is presently the material of choice for offshore structures. Therefore, more testing is required to establish safe design criteria.

1.2 Purpose and Scope of the Investigation

The objective of this investigation was to determine the influence of hydrostatic pressure on the strength of concrete subjected to static and fatigue loading. The materials for the high strength LWA concrete used in this investigation were supplied by the Norwegian Contractors. Test specimens were cores with a diameter of 5.75 in. (146 mm) and a length of approximately 11.8 in. (300 mm). Based on tests of cores, the concrete had a 28-day compressive strength of 8580 psi (59 MPa).

A series of static tests investigated the influences of submergence, rate of loading, and confining pressure. Loading rates of 0.036, 36, or 36000 psi/s (250 Pa/s, 250 KPa/s, or 250 MPa/s) were imposed on unjacketed specimens in a triaxial cell; cores were tested dry at atmospheric pressure or submerged in water with confining pressures of 0, 500, or 1000 psi (0, 3.5, or 7.0 MPa). A series of fatigue tests employed the same moisture and confining pressures as the static test program. In the fatigue tests, the maximum stress level was 60, 70, or 80 percent of the static compressive strength while the minimum stress level was 5 percent of the maximum stress level. With the exception of three cores loaded at a frequency of 0.1 Hz, tests were conducted at 1.0 Hz.

CHAPTER II

LITERATURE REVIEW

2.1 Effect of Rate of Load on Static Strength

It is generally agreed that for concrete the lower the rate of loading, the lower the measured strength. One explanation for this trend is that high rates of loading permit more subcritical crack growth to occur. Also, slower rates of loading cause an increase in creep which will increase the strain. Since the failure of concrete has been found linked to a limiting strain, creep strain will reduce the stress at failure.

Neville [1,2] plotted the results of three experiments performed by several investigators. These results, shown in Figure 1, indicate that increasing the rate of load on a compressive specimen from 0.1 psi/s to 1×10^7 psi/s (0.04 MPa/min to 4×10^6 MPa/min) doubles the apparent strength of concrete. Neville [1,2] included the results by Evans, who found no influence of the rate of load for rates below 1×10^4 psi/s (4 GPa/min); the results by Evans have not been verified by others and are therefore not considered useful.

2.2 Permeability of Concrete

Permeability is the rate of flow of a fluid through a porous medium. The coefficient of permeability of concrete is usually calculated from Darcy's law, which can be expressed as follows:

$$Q = AKi$$

where

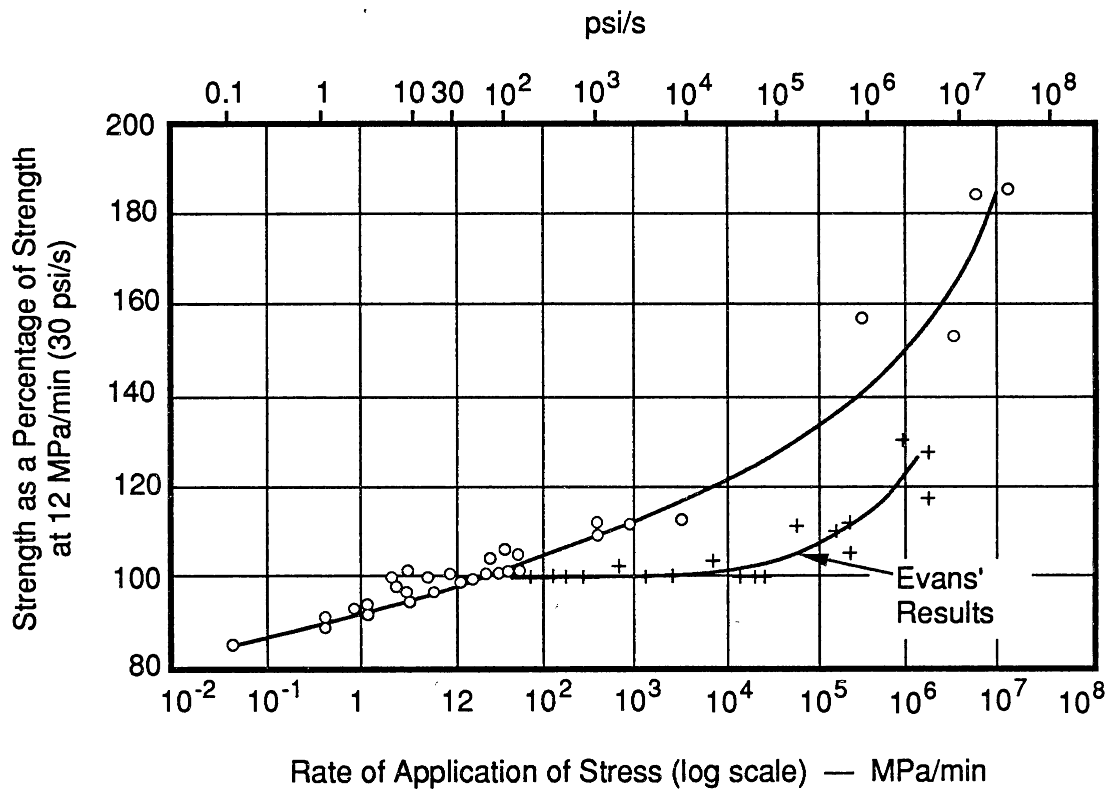


Figure 1. Influence of the Rate of Application of Load on the Compressive Strength of Concrete [1]

K = coefficient of permeability (m/s);

Q = rate of flow of water (m^3/s);

A = cross-Sectional area of flow(m^2) ; and

i = hydraulic gradient.

The coefficient of permeability, K, is usually measured in a laboratory by applying a water pressure to the upper face of a concrete specimen and then observing the rate of flow through the concrete.

High strength and lightweight aggregate concretes have a very low permeability. This comes from the effect of a low water/cement ratio reducing the capillary porosity on which the coefficient of permeability is strongly dependent.

Butler [3] has summarized the permeability measurements obtained by several investigators. The specimens were cylinders with diameters varying from 150 to 450 mm and lengths from 50 to 450 mm. In these tests, the applied pressure varied from 0.35 to 2.80 MPa, the water/cement ratio ranged from 0.35 to 0.84, and test duration varied from 24 to 408 hrs. With this range of parameters, the coefficients of permeability varied between 2×10^{-12} to 4436×10^{-12} m/s.

Haynes [4] and Buenfeld and Newman [5] found that concrete has a lower permeability in a seawater environment. They attributed the lower permeability to chemical reactions between the ions in seawater and the hydrated cement which resulted in crystallization products or chemical compounds that precipitate to block the pores in concrete.

2.3 Pore Pressure

2.3.1 General

Concrete is a porous material consisting of a complex system of capillary pores, gel pores, air voids, and aggregate pores. Permeability is strongly influenced by the presence of capillary pores which are relatively large and interconnected. Low strength concretes have many capillary pores while high strength concretes have few or none. Therefore, low strength concrete is more permeable than high strength concrete.

If a permeable concrete has become saturated with water and the internal pore pressure is in equilibrium with the surrounding pressure, the influence of pore pressure on granular stress is negligible. However, when a compressive load is applied, concrete can develop a positive pore pressure with respect to the confining pressure. This positive pore pressure will initially help to resist the compressive load by reducing the stress in the granular structure; at the same time the positive pore pressure will induce tensile stresses in the granular structure in directions perpendicular to the imposed load. The tensile stresses will promote the formation of tensile cracks. As these cracks develop, the corresponding volumetric dilation will cause the pore pressure to become negative with respect to the confining pressure. If loading is continued to failure, negative pore pressure would tend to increase the apparent strength of the concrete.

The above discussion was for a permeable, saturated concrete loaded at a rate too fast for water to flow and maintain equilibrium of pressure. However, when a high strength LWA concrete is involved, global saturation will be difficult to achieve, because there will be few, if any, capillary voids. Therefore, in a high strength LWA concrete, only small zones are likely to become saturated and develop pore pressures. However, if loading sufficient to produce cracking is imposed, water may reach the interior of the concrete and exert pressure at microscopic zones where cracks are propagating.

2.3.2 Boundary Porosity of Concrete

Richart et al. [6] performed tests on jacketed, dry concrete specimens with a surrounding pressure and found the compressive strength increased. They found the increase was equal to 4.1 times the confining pressure. This coefficient will be termed the "coefficient of increase" by the current author. Hanson [7] performed similar tests on lightweight concrete and found the coefficient of increase to be approximately 2.0.

Terzaghi [8] has discussed the fracture of concrete and rocks—materials which he terms "quasi-isotropic"—subjected to uniaxial compressive load. He observed that while the average stress acting on a plane parallel to load must be zero, local tensile and compressive "scatter stresses" are present. When the compressive load reaches a level where the tensile scatter stresses are equal to the bond strength of the material between grains, cracks begin to form approximately parallel to the direction of load. He noted that the strength of grains was not influenced by confinement and was much greater than the bond between grains.

Terzaghi concluded that when samples were tested with a watertight membrane, a confining pressure produced a precompression in the bond regions between grains and cracking would not occur until the tensile scatter stresses were equal to the sum of prestress and tensile bond strength. However, when a fluid under an applied pressure enters the voids, the compressive stress which the bond material carries will be reduced. This will cause the compressive strength of the specimen to decrease with respect to that for a specimen with a watertight membrane and no internal fluid pressure. The magnitude of this compressive strength decrease is dependent on the degree of continuity of the intergranular bond.

To quantify the degree of continuity, he defined "boundary porosity" as "the ratio of that part of the area of the potential surface failure which is in contact with the interstitial liquid to the total area of this surface." He reported no influence of confining pressure when concrete specimens were tested without membranes. Although he did not give experimental details, the concrete was apparently of low strength and loading was applied slowly. His conclusion was that the boundary porosity of concrete was nearly 100 percent.

Tests to determine the boundary porosity ratio have been performed by numerous investigators. Butler [3] has summarized some of these investigations.

Some results determined the boundary porosity ratio to be approximately equal to unity, while others have found it to be considerably less than unity.

2.4 Static Strength in a Water Environment

Haynes and Highberg [9] reviewed past studies on submerged, pressure resistant, concrete structures. The review indicated that uniaxial compressive strength of partially saturated concrete is 10 to 28 percent lower than concrete in a field dry condition.

Haynes and Highberg tested fourteen 6- by 12 in.(152-by 305 mm) concrete cylinders exposed to seawater and hydrostatic pressure. The cylinders were made of ND concrete with a water/cement ratio of 0.51 and a 28-day compressive strength of 7390 psi (45.7 MPa). The cylinders were tested in uniaxial compression while exposed to simulated ocean depths of 1, 500, and 20000 ft. (0.3, 152, and 6100 m). Before testing, the specimens were fog cured for 128 days and the specimens to be tested at pressure heads of 500 and 20000 ft were then placed in pressure vessels containing seawater and pressurized to 225 and 8900 psi (1.55 and 61.4 MPa), respectively, for a period of 48 to 65 days. Also, the specimens were pressurized and depressurized four to six times to possibly increase the rate and quantity of water absorption into the concrete. Prior to testing, the specimens were removed from pressure vessels and placed in the test chamber where the pressure head was reimposed for 18 hours.

The results of tests conducted with pressure heads of 1- and 500-ft depths showed increases in strength of 5 and 6 percent, respectively, when compared with control specimens that were continuously fog-cured. The authors did not consider these strength increases to be statistically significant. Analysis of the water absorption for specimens at a pressure head of 1 ft showed approximately zero absorption. The specimens tested at a pressure head of 500 ft had a low degree of saturation when compared to specimens tested at a pressure head of 1 ft.

The specimens tested at a pressure head of 20000 ft showed a 10 percent decrease in strength which was considered statistically significant by the authors. The cylinders also had a high degree of water absorption when compared to the specimens tested at a pressure head of 1 ft. The authors assumed that most of the voids in the specimens were filled with seawater and attributed the 10 percent reduction in strength to saturation of the specimens. Haynes and Highberg apparently did not consider that the several cycles of pressurization and depressurization might have damaged the specimens and caused the reduction in strength.

Haynes and Underbakke [10] compared the compressive strengths of high and low strength concretes which were cured under the following conditions: in the deep ocean at a depth of 1830 ft (560 m) and at a temperature of 42^{o}F (6^{o}C); in a fog room at 73^{o}F (23^{o}C); and in a tank with circulating seawater at an average temperature of 66^{o}F (19^{o}C). The cylinders were 6 by 12 in.(152 by 305 mm) with 120 made of low strength concrete and 120 made of high strength concrete. Two frames, each containing 18 molds with low strength concrete and 18 molds with high strength concrete, were transported by helicopter to the ocean site and allowed to free fall to the ocean floor. The concrete was apparently still in a plastic state when subjected to seawater. Efforts to recover one frame at three months were unsuccessful. The other frame was recovered at an age of 10 months. The deep ocean specimens were then placed in pressure vessels for temporary storage and repressurized to a hydrostatic head of 1830 ft. At the time of test, the deep ocean specimens were removed from the storage vessels and placed in a large test chamber containing three portable compression testers. The specimens were either tested at a simulated ocean depth of 1830 ft, a pressure head of 6 ft (2 m), or at a pressure head of 6 ft after being cycled rapidly three times from zero to a pressure head of 1830 ft. The fog-cured specimens were tested in the laboratory environment, while the tank-cured specimens were tested under a pressure head of 6 ft.

The low strength concrete from the fog room had a 28-day compressive strength of 3300 psi (22.8 MPa) and the seawater-tank-cured concrete had a 28-day compressive strength of 3400 psi (23.4 MPa). At an age of approximately 11 months, compressive strengths of specimens cured in a fog room, in a tank with seawater, and in a deep ocean environment were similar. For the latter curing condition, there was no significant influence from the three pressurization procedures used at the time of test.

For high strength concrete, specimens cured in a fog room and in a tank with seawater exhibited similar strengths at 28 days. However, at an age of approximately 11 months, high strength concrete cured in a fog room was stronger than concrete cured in a tank with seawater or in a deep ocean environment. Comparing the strength of concrete cured in a fog room, concrete cured in a tank with seawater had a strength loss of 9 percent while the concrete cured in the deep ocean had a strength loss of approximately 14 percent. Again, the three pressurization conditions involved with the deep ocean concrete did not cause a significant influence. Haynes and Underbakke attributed the reductions in strength to the effects of saturation. Alternative explanations including different curing temperatures and chemical reactions between the paste and seawater were not considered.

Clayton [11] performed investigations on compressive, tensile, and flexural strengths of concrete specimens exposed to high water pressure. The specimens were made of ND concrete with a water/cement ratio of 0.65. The pressurization process consisted of placing half the specimens in pressure vessels filled with water which were pressurized at a rate of 2 MPa/min (290 psi/min) up to 58.8 MPa (8530 psi). This pressure was maintained for 6 days after which time the vessels were depressurized at a rate of 2 MPa/min (290 psi/min) back to zero. The concrete specimens were cured in water at least three months prior to testing.

The compression tests involved sixteen 100 mm cubes and twelve 90- by 90- by 300-mm prisms. The cubes had a compressive strength of approximately 39.5 MPa

(5730 psi), while the prisms had a compressive strength of 31.5 MPa (4570 psi). Pressurization caused compressive strength reductions of 13 and 25 percent for the cubes and prisms, respectively. Clayton found that 90- by 90- by 300-mm prisms tested in flexure exhibited a 50 percent loss in strength as a result of the pressurization cycle. The tensile strength tests were performed on 100- by 200-mm cylinders using the "gas pressure tension test" which was developed at the Building Research Station in London, England. He also found that the pressurization process caused an 85 percent reduction in tensile strength. This large loss of tensile strength, when compared to the loss in cube strength, suggests that extensive microcracking had taken place in the concrete along the coarse aggregate/mortar interfaces which was confirmed by microscopic examinations. This portion of the experiment was repeated using a maximum pressure of 3 MPa (440 psi) and reported a 10 percent reduction in tensile strength.

Clayton then performed follow-up tensile and flexural tests to determine the effect of pressurization, immersion period, and depressurization. Conclusions and observations made by Clayton in these tests can be summarized as follows:

1. The rate of pressurization has no effect unless carried out very slowly. If the pressurization rate is carried out very slowly, there is less loss of tensile strength.
2. In the tensile tests, there was a significant strength loss no matter how short the immersion period. However, the greatest loss in strength resulted from immersion periods greater than 90 minutes.
3. The depressurization rate did not crucially affect the tensile strength loss.
4. Flexural tests conducted while specimens were subjected to peak pressure (i.e., no depressurization) exhibited the same loss in strength as those tested after depressurization.

Bjerkeli [12] studied the influence of water pressure on the compressive strength of concrete. The specimens were made using two types of ND concrete and one LWA concrete. All test specimens were cylinders with dimensions of 100 by 250 mm. The 28-day cube compressive strengths for the normal density concretes were 25 MPa (3630 psi) and 80 MPa (11600 psi). The LWA concrete had a 28-day cube compressive strength of 65 MPa (9430 psi). A total of 101 specimens were tested with 69 of these exposed to water pressure and 32 used as references. Most of the specimens subjected to water pressure were tested unjacketed while some were covered with a watertight membrane for comparison. The cylinders were stored and tested under 0, 400, or 800 m seawater pressure head for 0, 1, 7, 28, or 56 days without a pressure cycle. All specimens were 216 days or older before testing to failure. Specimens were loaded at a strain rate of approximately 5×10^{-6} in./s.

The LWA concrete specimens exposed to a pressure head of 400 or 800 m for periods up to 56 days were stronger than those tested without the pressure head. Separate absorption tests showed 8 and 17 mm of penetration for 7 and 144 days of exposure to a pressure head of 800 m. The results indicate that the internal pore pressure in the LWA concrete did not reach equilibrium with the surrounding water after 56 days of exposure. As a result, there was partial confining action from the hydrostatic head. From the results of the LWA concrete specimens covered with a watertight membrane, it was determined that axial compressive strength had a coefficient of increase of approximately 2.0.

The high strength ND specimens exposed to a pressure for 1, 7, and 28 days exhibited an increase in strength. No increase in strength was observed for specimens exposed to a pressure head for 56 days. Separate absorption tests indicated complete penetration of water at a pressure head of 800 m would occur after 56 days of exposure to the pressure head. The strength of unsealed specimens made with low strength ND concrete exposed to a water pressure for 1 to 28 days was equal to the strength of specimens tested without pressure.

Waagaard et al. [13] studied the effect of confining pressure on the static strength of plain concrete cylinders. They studied high strength LWA concrete with a water/cement ratio of 0.41 and with a 28-day cube strength of 70 MPa. The test specimens were cored from a slip-formed structure and had dimensions of 100 by 300 mm. The ends of the specimens were surfaced by grinding and then stored in seawater until testing. Confining seawater pressures corresponding to water depths of 60 and 350 m (197 and 1148 ft) were used. Compression tests were performed with seven cylinders in an air dry condition, five unjacketed at a pressure head of 60 m, four unjacketed at a pressure head of 350 m, and three jacketed by a watertight membrane and subjected to a pressure head of 350 m. Prior to testing the samples underwent a soak period of about 20 hours.

Cylinders tested with a pressure head were stronger than those tested in a dry condition. The increase in strength was approximately four times the confining pressure for the unjacketed specimens and about five times the confining pressure for the jacketed specimens. Splitting tests after failure showed that the water had only penetrated 2 to 3 mm into the specimens. Therefore, the pore pressure in the cores did not fully develop.

The observed mean axial ultimate strain of the specimens at the 60-m water pressure was 3.6 percent while the mean value of the specimens at the 350-m water pressure was about 4.2 percent. Three cores were instrumented for transverse strain and showed a constant value of 0.25 for Poisson's ratio up to a load of roughly 70 percent of failure.

2.5 Fatigue Strength in a Water Environment

Fatigue may be defined as the phenomenon by which a material is caused to fail by the application of repeated loads that are not large enough to cause failure in a single application. This means that under cyclic loading permanent structural damage takes place in the concrete. Fatigue data are usually presented graphically in the form of an S-N diagram. For concrete this diagram is normally constructed with S, the ratio of peak stress to static ultimate strength, plotted versus the logarithm of N, the number of cycles to failure. Many investigations on the fatigue strength of concrete in air have been performed and papers summarizing the results of these studies are available [14, 15].

Viswanathan [16] performed fatigue tests on 3- by 6-in. (76- by 152-mm) and 4-by 8-in. (102- by 203-mm) saturated mortar cylinders. After casting the specimens were cured in a fog room for 125 to 236 days. The specimens were then oven dried to a constant weight at a temperature of 95 to 100°C and back pressure saturated with the aid of a vacuum chamber. Based upon mix proportions and weight data, Viswanathan determined that the specimens had achieved an average degree of saturation of about 97 percent. Specimens were tested either submerged in water or in air with a moist surface. Sinusoidal fatigue loads were applied at a frequency of 1 or 10 Hz. In some tests brief rest periods were interspersed in the loading.

Viswanathan found that fatigue strength of saturated mortar was much less than dry concrete. He also found that rest periods did not influence fatigue behavior and that larger specimens exhibited a slightly lower fatigue strength than smaller specimens. Also, Viswanathan found that specimens tested at a frequency of 10 Hz had longer fatigue lives than those tested at 1 Hz.

Leeuwen and Siemes [17] conducted fatigue tests on plain concrete under submerged conditions. The main parameters in the study were storage time in water, state of saturation, and effect of frequency. Although the distribution of this report is presently restricted, the work has been summarized by Waagaard [18]. He reports that submerged concrete had a lower fatigue life than air-dried concrete; that an extended storage time in water resulted in a shorter fatigue life; and using a lower frequency of loading lowers the fatigue life.

Siemes [19], citing information obtained in another confidential study [20], reports that concretes with low water/cement ratios, high cement contents, silica fume, and lightweight aggregate have a lower fatigue strength when compared to gravel concrete. He also stated that little is really known about the fatigue properties of LWA concrete.

Muguruma et al. [21] conducted low-cycle fatigue tests on two high strength ND concretes with water/cement ratios of 0.26 and 0.40 which had compressive strengths of 1210 kgf/cm^2 (119 MPa) and 598 kgf/cm^2 (59 MPa), respectively. All specimens were 75 by 150 mm (3- by 6 in.) and initially cured in water; at 8 weeks two-thirds of the cylinders were removed from the water and allowed to air dry for 6 to 10 weeks. One-half of the air-dried specimens were tested in air; the remainder were resubmerged and tested in water. The authors did not state if a soak period was employed before testing. Cylinders cured in water until the time of test were removed from the water and tested in air. For cylinders with the lower water/cement ratio, maximum stress levels of 70, 80, and 90 percent of the static compressive strength were used; for cylinders with the higher water/cement ratio, maximum stress levels of 80, 90, and 96 percent were employed. For both concretes the minimum stress level was 5 percent of the static compressive strength. Most specimens were tested at a frequency of 1 Hz while a few were loaded at 10 Hz. Testing of the specimens began when they were 14 to 18 weeks old.

Specimens which were air dried and tested in air had greater fatigue lives than specimens tested in air after water curing or in water after air drying; the benefits of air drying and testing in air were most pronounced for specimens made with the higher water/cement ratio. For specimens reintroduced to water after several weeks of air drying or exposed to air drying after continuous curing in water, it is likely that the static strength will be strongly time-dependent. Consequently, stress levels reported for these specimens may be inaccurate.

More recently, Muguruma and Watanabe [22] have extended this research by incorporating acoustic emission technology to detect microcracking. They employed four concrete mixes. Specimens were cured in water for 5 weeks, air dried for an unspecified period, then subjected to fatigue loading, either in air or submerged in water. They monitored acoustic emissions and the maximum longitudinal strain during each cycle of loading. The maximum longitudinal strain increased with cycles of load. They noted that when the cumulative number of emissions were plotted against the maximum longitudinal strain, there was a region of linearity. Shortly before failure, the longitudinal strain increased much more rapidly than the emission count. For a given type of concrete tested at a specific stress level and moisture condition, Muguruma and Watanabe noted that the linear relationship ended at approximately a constant "limiting" strain. Unfortunately they did not determine if the limiting strain was constant for other stress levels.

Waagaard et al. [13] performed fatigue tests on 38 cored cylinders: 18 specimens were tested at a confining pressure head of 60 m with 6 having a single crack normal to the direction of the load in the middle of the cylinder; 17 specimens were tested at a pressure head of 350 m. Of these, 6 were solid, 3 were precracked, and 8 were covered with a watertight membrane. The remaining 3 cores were tested in a dry condition. The maximum load levels ranged between 50 to 70 percent of the static compressive strengths determined under the same pressure and exposure conditions as employed in the fatigue tests. The minimum load level was 5 percent of the static compressive strength. The loading was sinusoidal and applied at a frequency of 1 Hz.

The results showed that precracked and noncracked solid specimens had similar fatigue lives; there seems to be slightly more scatter of data for the precracked specimens. The presence of a membrane appeared to be beneficial to fatigue life for tests conducted at a stress level of 0.6 and at a pressure head of 350 m; however, this trend was not noticeable for tests at a stress level of 0.7. Increasing the pressure head from 60 to 350 m increased the static strength from 82 to 92 MPa. However, fatigue tests conducted with stress levels referenced to these static strengths revealed that specimens tested with higher pressure head had lower fatigue lives; the benefit of confinement was therefore less. Specimens which were dried for one week prior to test had greater strength than specimens tested submerged at a 60 m pressure head.

Petkovic et al. [23] studied the effect of different moisture conditions on the fatigue strength of concrete. Three different sizes of cylinders were used: 50 by 150 mm, 100 by 300 mm, and 450 by 1350 mm. Two ND concretes and one LWA concrete were tested with cylinder strengths of 55, 75, and 80 MPa (7000, 10900, and 11600 psi), respectively. Three moisture conditions were employed: specimens were stored and tested in air, stored and tested in seawater, or sealed and tested in air. The testing frequency was 1 Hz for the two smaller sizes and 0.5 Hz for the larger cylinders.

It was found that the fatigue life of the 450-mm diameter cylinders made with ND 75 MPa concrete was approximately 10,000 cycles for all three moisture conditions. For specimens with diameters of 50 and 100 mm, tests in water resulted in the lowest fatigue lives. When 100-mm diameter specimens were tested dry, the fatigue life of the ND concrete increased from several thousand cycles to several hundred thousand cycles while the fatigue life of the LWA concrete increased only slightly. For ND concrete, tests performed in a sealed condition yielded fatigue lives only slightly greater than tests in water; for LWA concrete, tests conducted in air and in a sealed condition gave almost identical fatigue lives.

Based on the review of available research, it is evident that the application of extremely high hydrostatic pressures can damage concrete. However, for hydrostatic heads corresponding to water depths of a few hundred meters, no damage is encountered. If concrete is unsaturated and directly exposed to a moderate hydrostatic pressure, partial confinement will increase the static strength by an amount somewhat less than when a watertight membrane is present. For unjacketed specimens, the benefits of partial confinement decrease during fatigue tests and may therefore be time-dependent.

Some researchers have presented data from the point of view that concrete experiences a strength loss when tested in water rather than a strength gain when permitted to dry. Evidence suggests that moisture state is more significant for an ND concrete than an LWA concrete. For high strength ND and LWA concretes, data indicated LWA concretes were more resistant to water penetration and therefore more resistant to saturation.

CHAPTER III

EXPERIMENTAL PROGRAM

3.1 Specimen Preparation

3.1.1 Mix Design

The high strength lightweight aggregate concrete used in this investigation was produced using materials provided by the Norwegian Contractors. The properties of the portland cement are given in Table 1; the cement was roughly equivalent to an ASTM type II cement. The coarse aggregate was "Liapor," an expanded clay aggregate supplied in two sizes: 4 to 8 mm and 8 to 12 mm (0.16 to 0.32 in. and 0.32 to 0.63 in.). The fine aggregate was a natural sand; the sand had a bulk specific gravity (SSD) of 2.68, an absorption of 0.6 percent, and a fineness modulus of 3.17. The sand was slightly coarser at the No. 8 sieve and slightly finer at the No. 100 sieve than the ASTM C33 gradation. The mix proportions on a weight basis were 1.00:1.69:0.75:0.94 of cement, fine aggregate, 4 to 8 mm and 8 to 16 mm aggregate, respectively. The concrete mix had a water/cement ratio plus condensed silica fume equal to 0.43; however, the coarse aggregate which was batched dry, absorbed significant water making the net water/cement ratio indefinite. To evaluate the approximate quantity of water which might be absorbed by the Liapor aggregate immediately after water was added to the mix, a small quantity of aggregate was immersed in water and its weight in water was monitored. The results of this test are shown in Figure 2. From Figure 2, it is evident that substantial water will be absorbed during mixing and placing operations. The mix also contained a quantity of silica

TABLE 1. PORTLAND CEMENT PROPERTIES

Oxide/Compound Analysis, %	
SiO_2	22.7
Al_2O_3	3.7
Fe_2O_3	2.8
CaO	64.1
MgO	1.6
SO_3	2.7
C_3S	51.4
C_2S	26.4
C_3A	6.1
C_4AF	8.4
Available Alkalies, %	
Na_2O	0.20
K_2O	0.57
Na_2O Equivalent	0.58
Fineness, m^2/kg	388
Mortar Strength, psi	
1 day	2570
3 days	3830
7 days	4860
28 days	6490

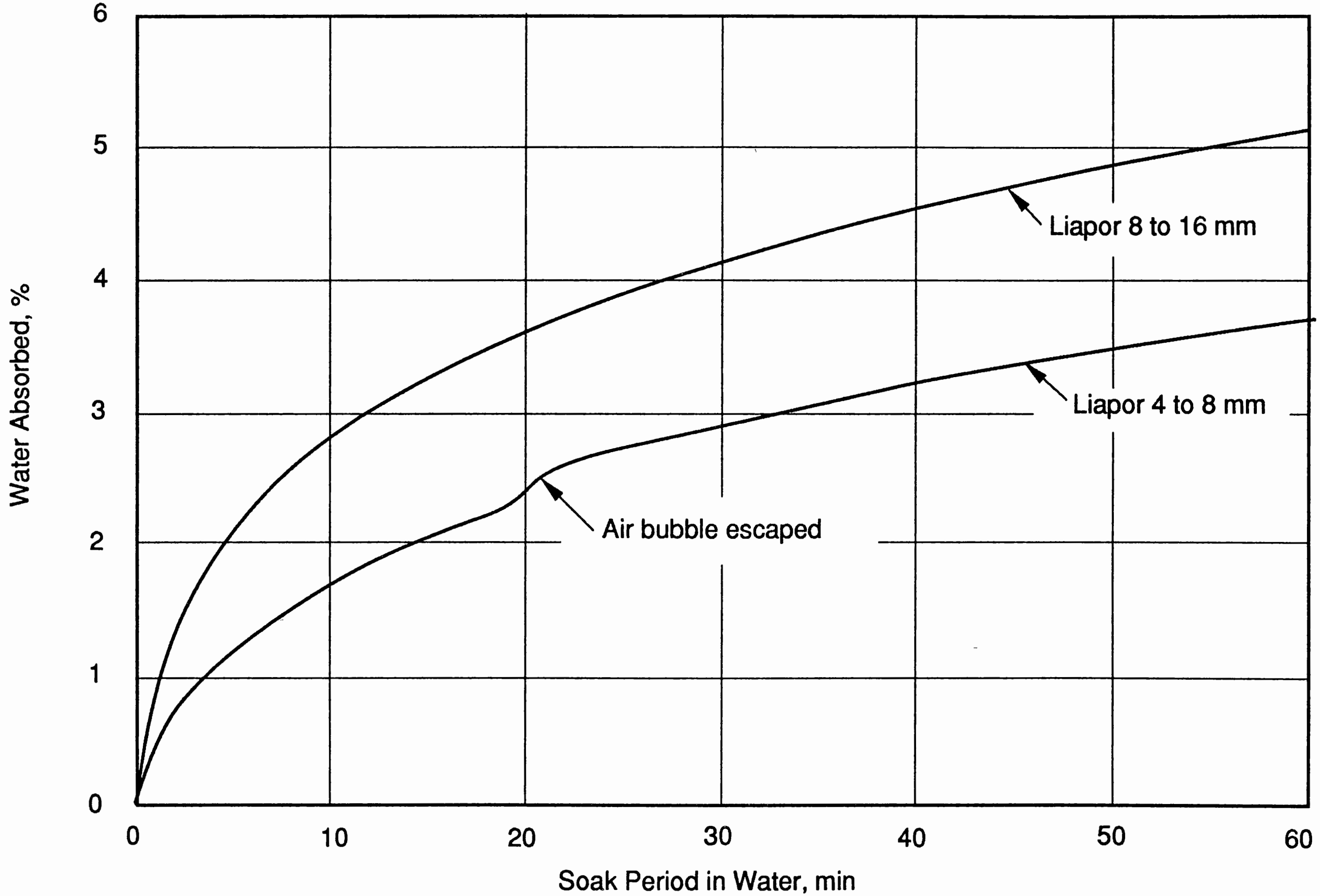


Figure 2. Water Absorption of Liapor Aggregate

fume equal to 3.9 percent of the weight of cement as well as air entraining and high range, water reducing admixtures.

3.1.2 Mixing, Consolidation, and Curing

The concrete tested in this study was cast from a single batch mixed in a pan mixer with a capacity of 1.5 yd^3 (1.1 m^3). Casting was performed at a precast concrete plant in Oklahoma City, Oklahoma. The dry materials were mixed continuously for 1 minute; then 95 percent of the water was added and mixed for 15 seconds. Next, the rest of the water and admixtures were added and the concrete was mixed for 2 minutes. Visually the concrete appeared to have a slump with the desired range of 7 to 9.5 in. (180 to 240 mm).

After the concrete was mixed, it was discharged into a bucket of a front-end-loader which placed the concrete into a transport truck which delivered the concrete to the casting yard approximately 0.25 mi (0.4 km) from the batch plant. The concrete was then discharged into a concrete bucket which was carried by an overhead crane to a large casting bed. A 1- by 5- by 7-ft (0.31- by 1.52- by 2.14-m) slab was cast in two lifts using wooden forms attached to the casting bed. Each lift was vibrated using external vibrators attached to the casting bed. Although the mix appeared to have the desired slump when discharged from the mixer, it stiffened noticeably during the transportation and placing operations. When the second lift was being poured, the slump appeared to be approximately 6 in. (150 mm). The remaining concrete which had been placed in a wheelbarrow stiffened so rapidly during strike-off of the slab that it was not possible to measure the air content or cast standard control cylinders. Approximately 45 minutes had elapsed since discharge from the mixer. The slab was then cured under plastic for 24 hr before transporting to the Civil Engineering Laboratory. In the laboratory it was cured under wet burlap and plastic until coring of the slab was completed.

3.1.3 Test Specimens

Cores were removed from the slab using a standard core drill with either a 6- or 4-in. bit. Cores were not taken closer than 1 in. (25 mm) to the formed surface. The larger bit produced cores with a 5.75-in. (146 mm) diameter, while the smaller bit yielded cores with a 3.75-in. (95 mm) diameter. An effort was made to obtain the largest possible number of large cores; smaller cores were only taken when regions of the slab would otherwise be wasted. Only 5.75-in. diameter cores were used in this study.

Ends of the cores were prepared for the tests by first removing major surface irregularities with a concrete saw. The ends of the cores were then ground smooth using a surface grinder equipped with a diamond wheel.

Most specimens were stored in a fog room until time of testing. For specimens which were to be tested in an air-dry condition, two drying procedures were used. Specimens which were to be tested under static load were dried at a time when the ambient humidity in the laboratory was quite high. Therefore, they were placed in a container above a saturated salt solution which maintained a relative humidity of 50 percent and dried for three months. Air-dried specimens tested under fatigue loading were exposed to ambient laboratory air for two months. Drying occurred during the late fall when the laboratory humidity was usually between 40 and 45 percent.

3.1.4 Compressive Strength and Core Densities

Based on the average of two core tests, the compressive strength at 7 days was equal to 7640 psi (52.7 MPa) and at 28 days was equal to 8580 psi (59.2 MPa). After the ends of the cores were surfaced and just before testing began, the density of each core was determined. The average density was 120.02 pcf (1922.54 kg/m^3) with a standard deviation of 0.88 pcf (14.10 kg/m^3). Specimens with a density outside one standard deviation from the mean were excluded from the test program.

3.2 Test Setup

3.2.1 Test Equipment

Loads were applied by a servo-controlled material test system with a capacity of 600 kip (2700 kN). The columns, cross-head, and load cell were designed for a capacity of 1100 kip (4900 kN), resulting in an unusually stiff load frame. A triaxial cell was employed which could accommodate cylinders as large as 6 by 12 in. The cell was rated for 10,000 psi (69 MPa) and was equipped with a hydraulic balance feature to offset the hydrostatic pressure acting against the loading ram. Because the hydraulic balance mechanism was for oil service, a piston-style accumulator rated for water was used to separate the oil going to the balance unit from water being delivered to the test chamber of the triaxial cell.

Deaired water was supplied to the triaxial cell using an air-over-fluid pump rated for water service. The water could be delivered directly to the cell when initially filling the chamber and bringing the cell to pressure or through a water-rated, double acting, hydraulic cylinder used to monitor the movement of water into the cell during tests. Some specimens were tested submerged in water with negligible pressure in the cell. For these tests, a pressure reducer was installed between the hydraulic cylinder used to monitor water movement and the triaxial cell. The pressure reducer limited cell pressure to approximately 15 psi (100 kPa).

A microcomputer was used to control the test system and to acquire data. During static tests the computer programmed a function generator to generate the proper voltage ramp. In fatigue tests the computer continually monitored the minimum and maximum loads and adjusted the function generator output as necessary.

Data acquisition was accomplished with a 4-channel A/D board with 12-bit resolution; all channels were sampled simultaneously. In addition to load applied to the triaxial cell, the A/D board monitored longitudinal and transverse strain, and volume of water entering the triaxial cell during a test. An overall schematic of the test setup is given in Figure 3.

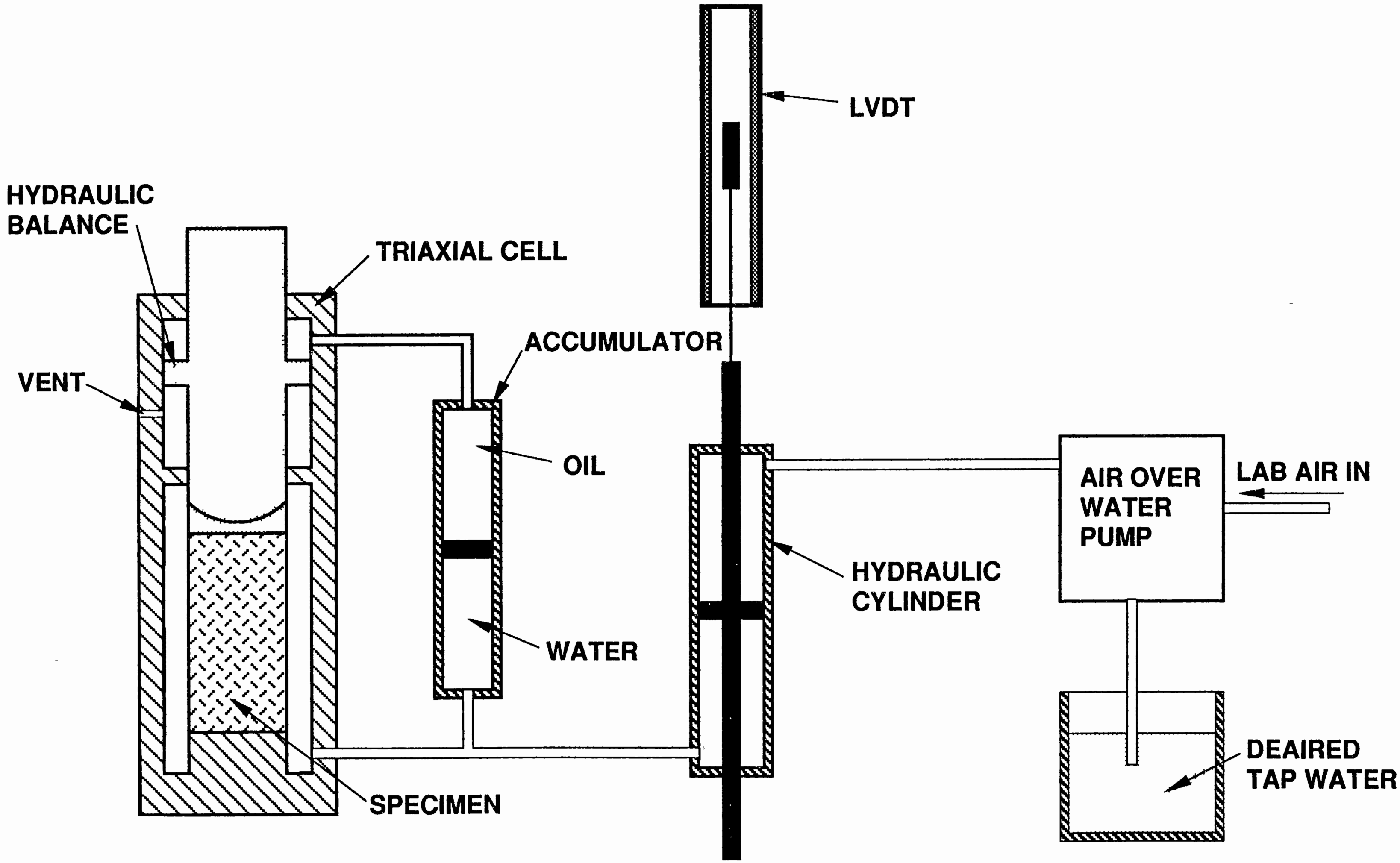


Figure 3. Schematic of Test Setup

The longitudinal strain was obtained using two LVDT transducers mounted diametrically opposed on the specimen. Brackets supporting the transformers and core rods were glued to the surface of the test cylinders. The transverse strain was also obtained using two LVDT transducers which were mounted diametrically opposed at midheight on the cylinder. The core rods for these LVDT transducers were connected to an aluminum ring which was supported at three locations by short lengths of stainless steel shim stock oriented at 45^{o} to the axis of the cylinder. When load produced a change in radius of a test specimen, the resulting horizontal motion of brackets glued to the core was transferred by the pieces of shim stock into equal vertical motion of the aluminum ring. The signals from the two pairs of LVDTs were averaged before being transmitted to the A/D board.

The core rod of the LVDT used to measure the volume of water entering the triaxial cell was simply connected to one of the piston rods of the doubleacting hydraulic cylinder mentioned above. The piston of the cylinder served as a reference plane in the water line between the air-over-water pump and the triaxial cell.

3.3 Test Program

All concrete tested in this program was well cured, high strength, LWA concrete. Two series of tests were performed in this investigation. In the first series, static tests were conducted to evaluate the influences of submergence, rate of loading, and confining pressure. In the second series, fatigue tests investigated the effects of submergence, confining pressure, and test frequency. All specimens were tested without jackets inside the triaxial cell.

In future tests, which will employ acoustic emission transducers attached to the specimens,1/16-in. (25/16 mm) thick disks of delrin plastic will be placed between the ends of specimens and the triaxial cell. The presence of delrin will cause much of the acoustic energy originating outside the specimens to be reflected and improve the ability to detect microcracking. To permit better comparison of results obtained in the total test program, these disks of delrin were employed in this study. New disks were used for each test.

In exploratory tests, several cores were placed in pressure vessels and subjected to a hydrostatic pressure of 7 MPa. A small uptake of water was observed during the first few hours of pressurization. Based on these observations, before the initiation of static or fatigue tests, a 24-hr soak period was observed after the application of confining pressure. For cores tested dry or submerged without a confining pressure, no soak period was employed.

In the two series of tests, specimens were in air after approximately three months of drying or in water at a pressure of 0, 3.5, or 7.0 MPa. For static tests, loads were applied at a rate of 250 Pa/s, 250 kPa/s, or 250 MPa/s. In the series of static tests, specimens had a minimum age of 603 days if tested in water or 771 days if tested in air. In the series of fatigue tests, specimens had a minimum age of 665 days if tested in water or 778 days if tested in air. In the series of fatigue tests, the maximum stress level was 60, 70, or 80 percent of the static compressive strength. This strength was the average obtained from four cores loaded at a rate of 250 kPa/s while submerged in water at zero pressure. The minimum stress level was 5 percent of the maximum stress level. Sinusoidal loading was applied at a frequency of 0.1 or 1 Hz. For the frequency of 0.1 Hz, only three tests with a confining pressure of 3.5 MPa and maximum stress level of 80 percent were performed. No air dry tests were conducted using a maximum stress level of 60 percent because of the excessive length of time which would have been required.

In addition to the main fatigue program discussed above, two specimens were tested with a confining pressure of 7.0 MPa and a maximum stress level of 70 percent with a florescent dye added to the water. Tests were terminated and specimens were removed from the triaxial cell prior to failure. The first specimen experienced a water uptake of approximately 6 $in.^3$ (0.1 liter) while the second specimen had an uptake of approximately 2 $in.^3$ (0.033 liter). The corresponding cycles for the two tests were

92,279 and 779 cycles, respectively. A split cylinder test was performed on the specimens to reveal the dye penetration.

CHAPTER IV

EXPERIMENTAL RESULTS

4.1 Static Tests

The strengths and moduli of elasticity obtained from static testing are given in Table 2. The modulus of elasticity, E, was calculated using a least squares linear regression for data acquired between 10 and 40 percent of the ultimate load. Stress-strain diagrams for each core are shown in Figure 4.

During tests conducted at 250 MPa/s, the small diameter hoses and style of pump used to supply water to the triaxial cell prevented significant uptake of water during the fraction of a second required for the test. Mechanical problems encountered at the beginning of the test program resulted in questionable uptake data for tests conducted at zero confining pressure and tests conducted at a load rate of 250 Pa/s.

After these problems were corrected, four cylinders were tested at a load rate of 250 kPa/s. Uptake data for these tests are shown in Figure 5. Also shown in Figure 5 are the volumetric strains computed from the LVDT data.

4.2 Fatigue Tests

The cycles to failure data are provided in Table 3. The quantities of water entering the triaxial cell during fatigue testing are shown in Figures 6 through 9. In these figures, the quantity of water entering the cell is expressed as a percentage of the volume of sample. No water entered the triaxial cell for the three specimens tested at a confining pressure of zero and maximum stress level of 70 percent.

There was a slight but measurable reduction in the axial stiffness of cores with an increase in the number of applied cycles of load. To illustrate this behavior, the

TABLE 2. RESULTS OF STATIC TESTS

	Dry in Air		Submerged in Water					
	0 MPa		0 MPa		3.5 MPa		7.0 MPa	
Rate of Loading	f_c', MPa*	E, GPa	f_c', MPa	E, GPa	f_c', MPa	E, GPa	f_c', MPa	E, GPa
250 Pa/S	55.2	17.5	49.7	15.5	59.3	22.4	59.3	20.7
	54.3	17.6	49.7	19.1	55.7	21.7	56.2	23.9
250 kPa/S	58.0	19.9	64.0	24.3	72.0	25.4	73.9	24.4
	61.0	20.3	55.9	22.5	67.9	26.0	75.2	25.8
			54.9	21.6				
			58.9	22.0				
250 MPa/S	75.2	20.4	78.1	26.3	89.0	26.1	95.7	24.9
	77.1	21.5	72.6	20.0	84.1	25.6	94.9	24.5

*To convert from Pa to psi, divide Pa by 6,895.

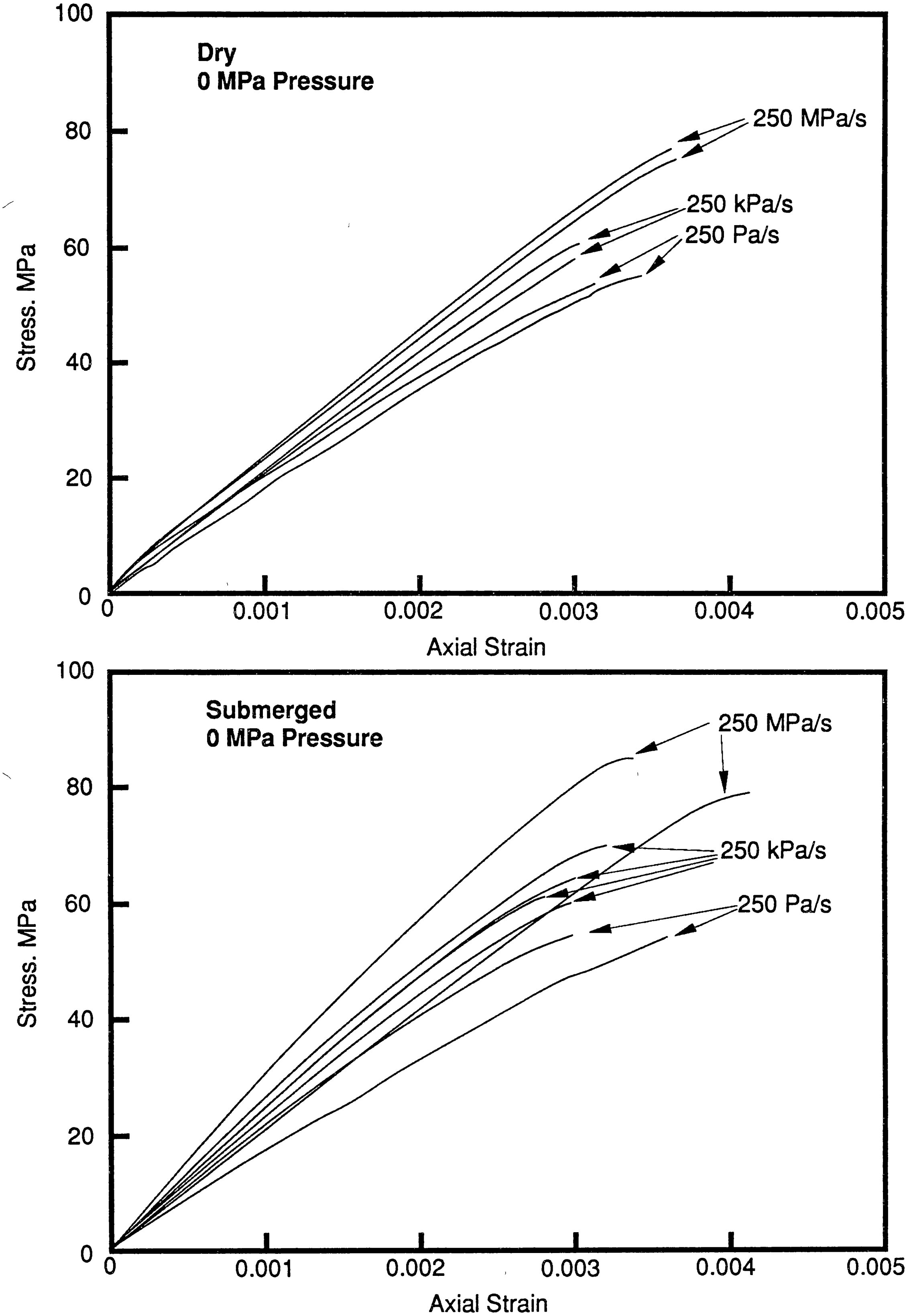


Figure 4. Static Stress-Strain Diagrams

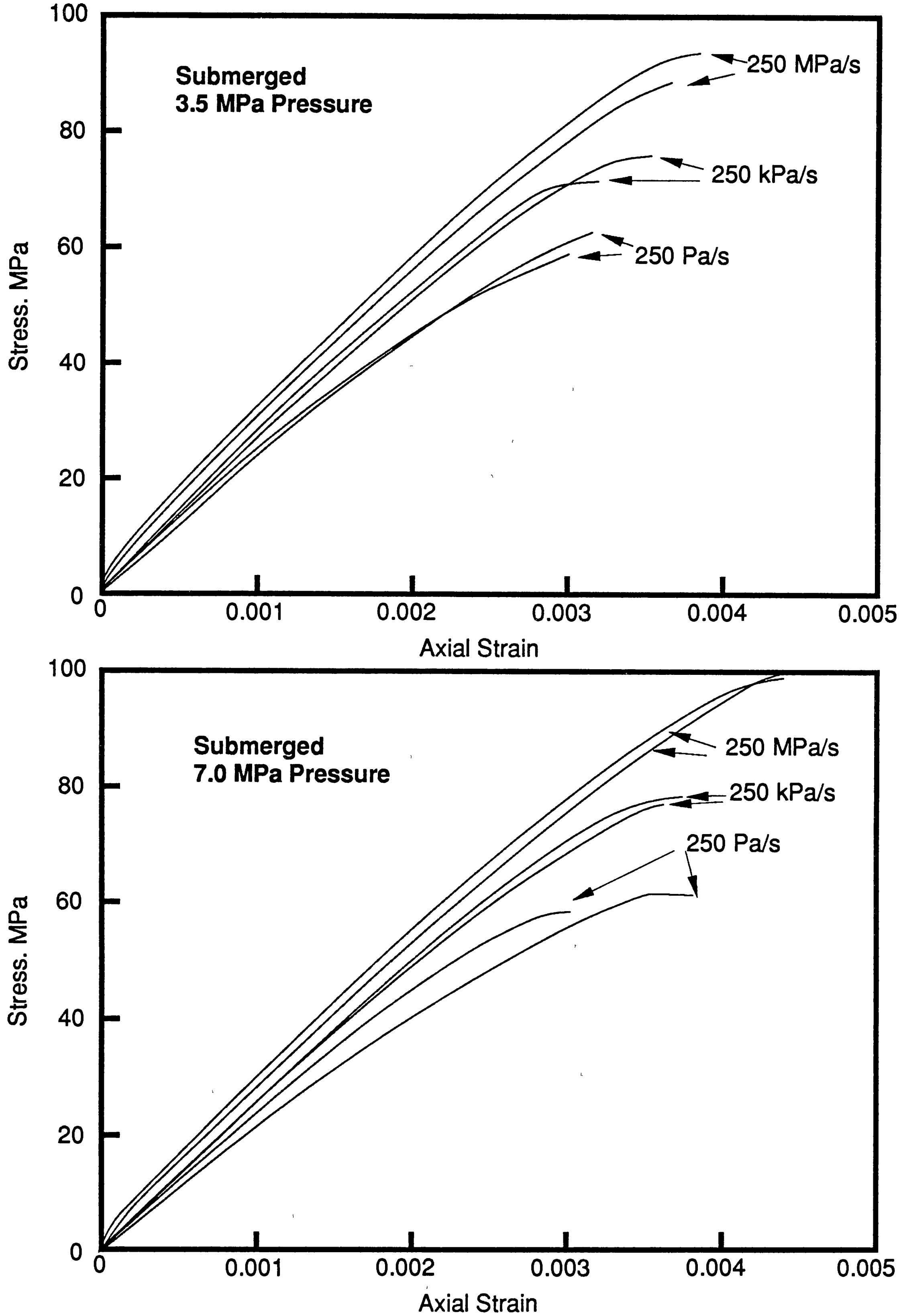


Figure 4. (Continued)

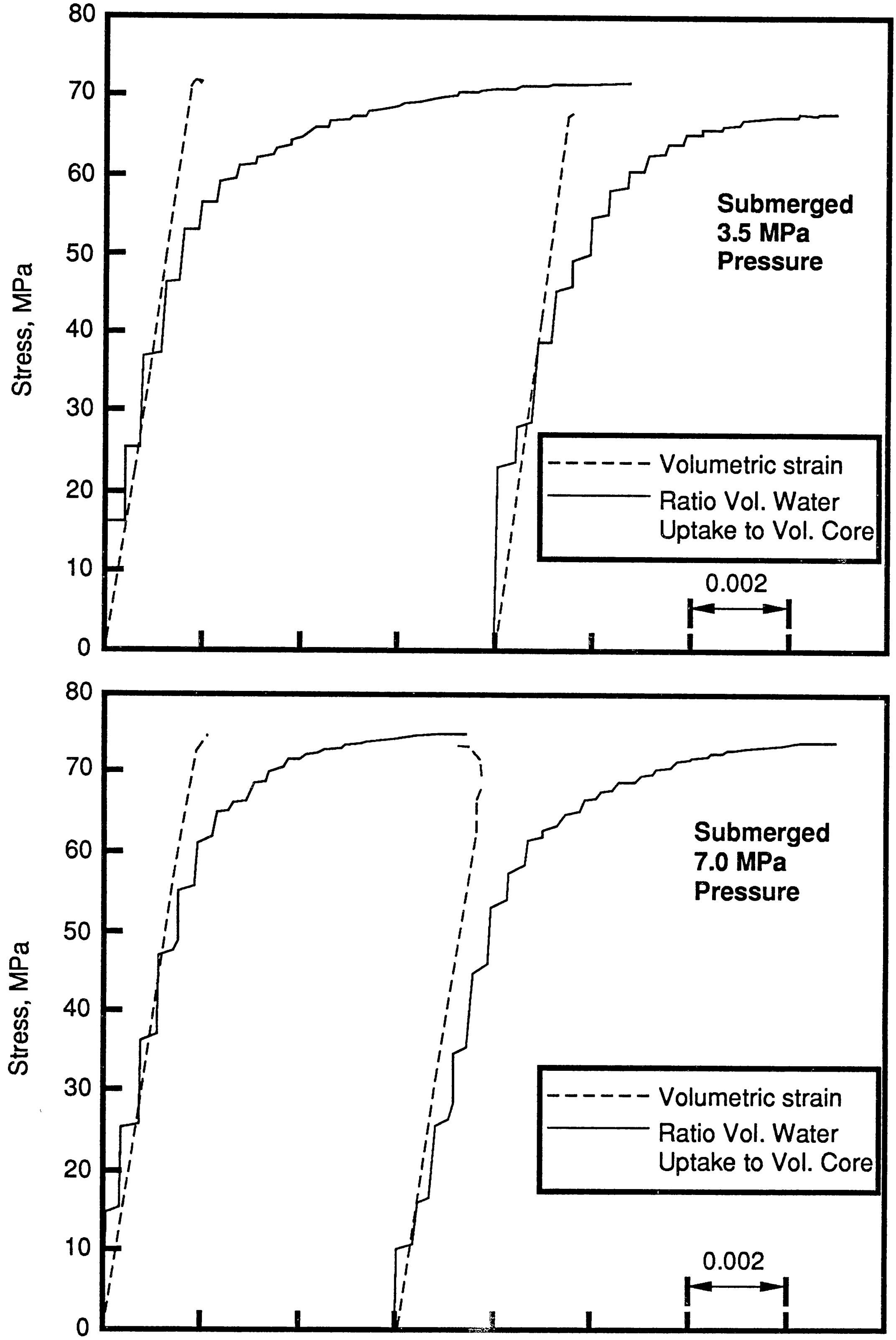


Figure 5. Volumetric Strain and Water Uptake at a Load Rate of 250 kPa/s

TABLE 3. RESULTS OF FATIGUE TESTS

Core No.	Triaxial State	Frequency, Hz	Stress Level, %	Confining Pressure, MPa	Cycles to Failure
32	Moist	1	60	0.0	213410
116	Moist	1	60	0.0	114186
96	Moist	1	60	0.0	343110
59	Moist	1	60	3.5	648180
61	Moist	1	60	3.5	882193
67	Moist	1	60	3.5	333127
55	Moist	1	60	7.0	404975
107	Moist	1	60	7.0	530324
43	Moist	1	60	7.0	182297
103	Moist	1	60	7.0	514980
54	Dry	1	70	—	738843
46	Dry	1	70	—	324275
66	Dry	1	70	—	474516
37	Moist	1	70	0.0	19164
99	Moist	1	70	0.0	14284
47	Moist	1	70	0.0	31936
81	Moist	1	70	3.5	40941
68	Moist	1	70	3.5	4678
93	Moist	1	70	3.5	90537
48	Moist	1	70	3 5	40456
57	Moist	1	70	7 0	118163
101	Moist	1	70	7.0	89502
112	Moist	1	70	7.0	104077
94	Dry	1	80	—	10626
51	Dry	1	80	—	14403
39	Dry	1	80	—	20471
111	Moist	1	80	0.0	4158
86	Moist	1	80	0.0	3659
27	Moist	1	80	0.0	1890
41	Moist	1	80	3.5	4946
97	Moist	1	80	3.5	6587
88	Moist	1	80	3.5	23558
74	Moist	1	80	7.0	10802
84	Moist	1	80	7.0	12372
90	Moist	1	80	7.0	12570
23	Moist	0.1	80	3.5	3432
17	Moist	0.1	80	3.5	2833
36	Moist	0.1	80	3 5	6118

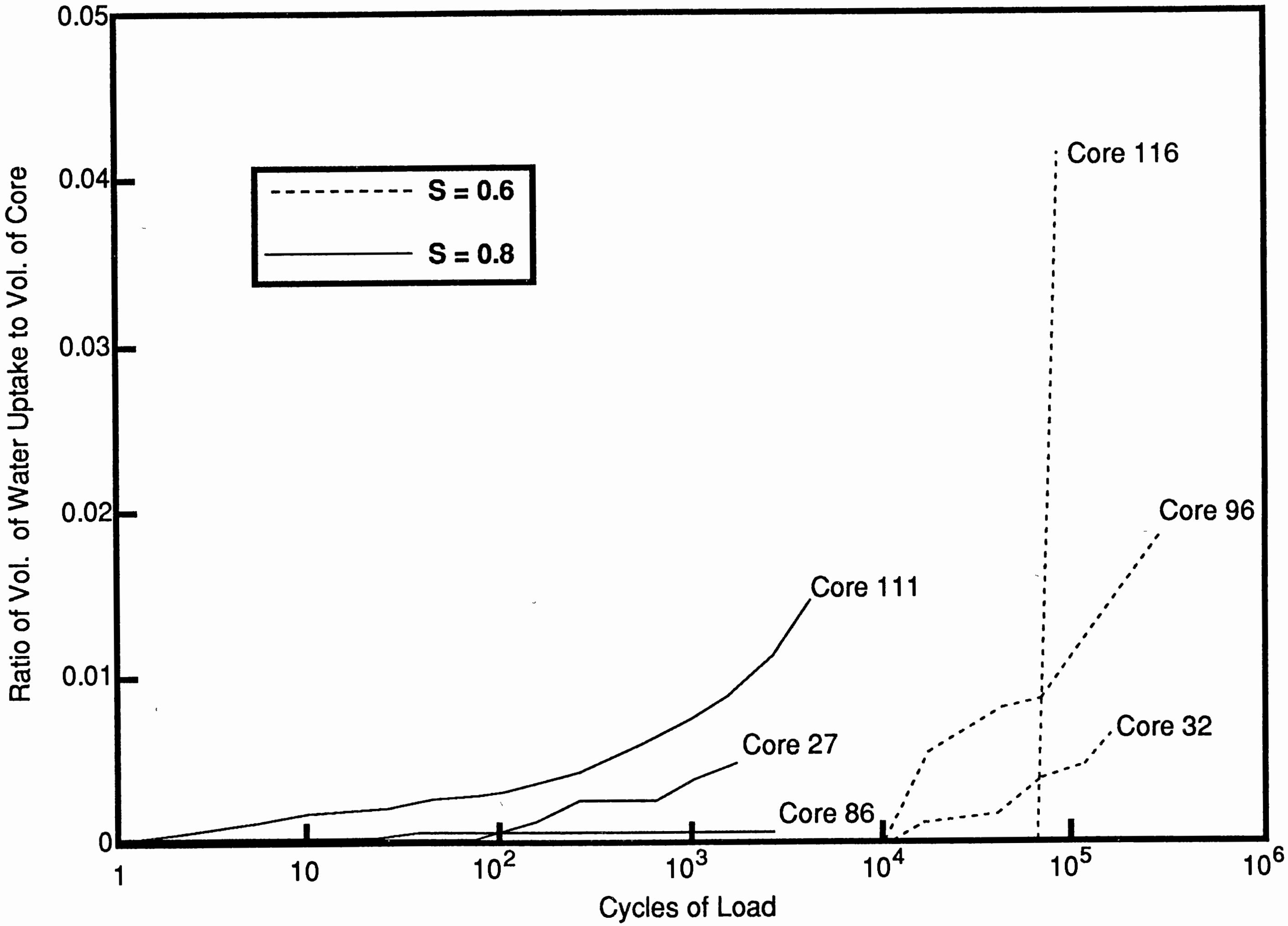


Figure 6. Water Uptake of Fatigue Tests With Confining Pressure of 0 MPa (Frequency of 1.0 Hz)

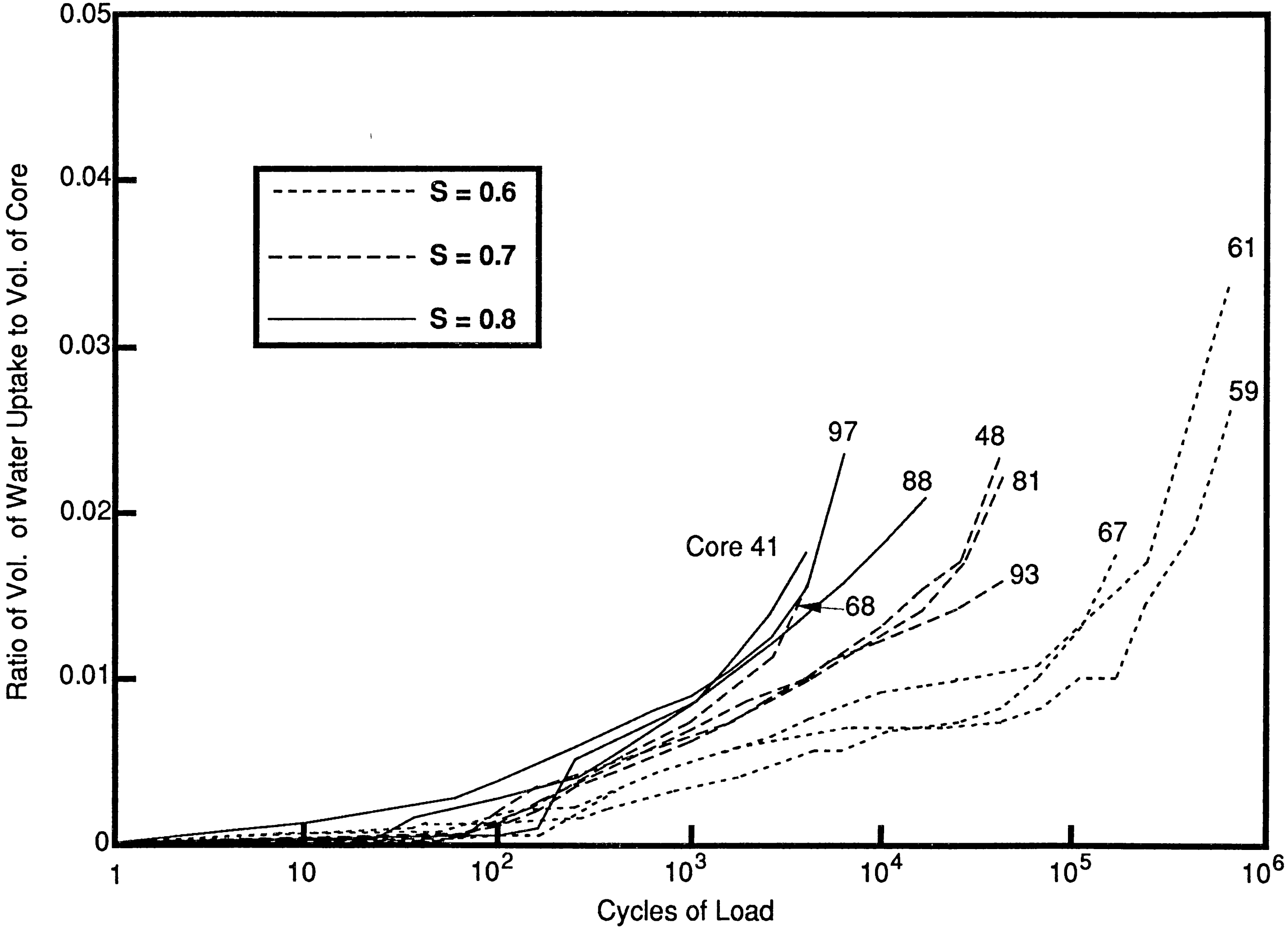


Figure 7. Water Uptake of Fatigue Tests With Confining Pressure of 3.5 MPa (Frequency of 1.0 Hz)

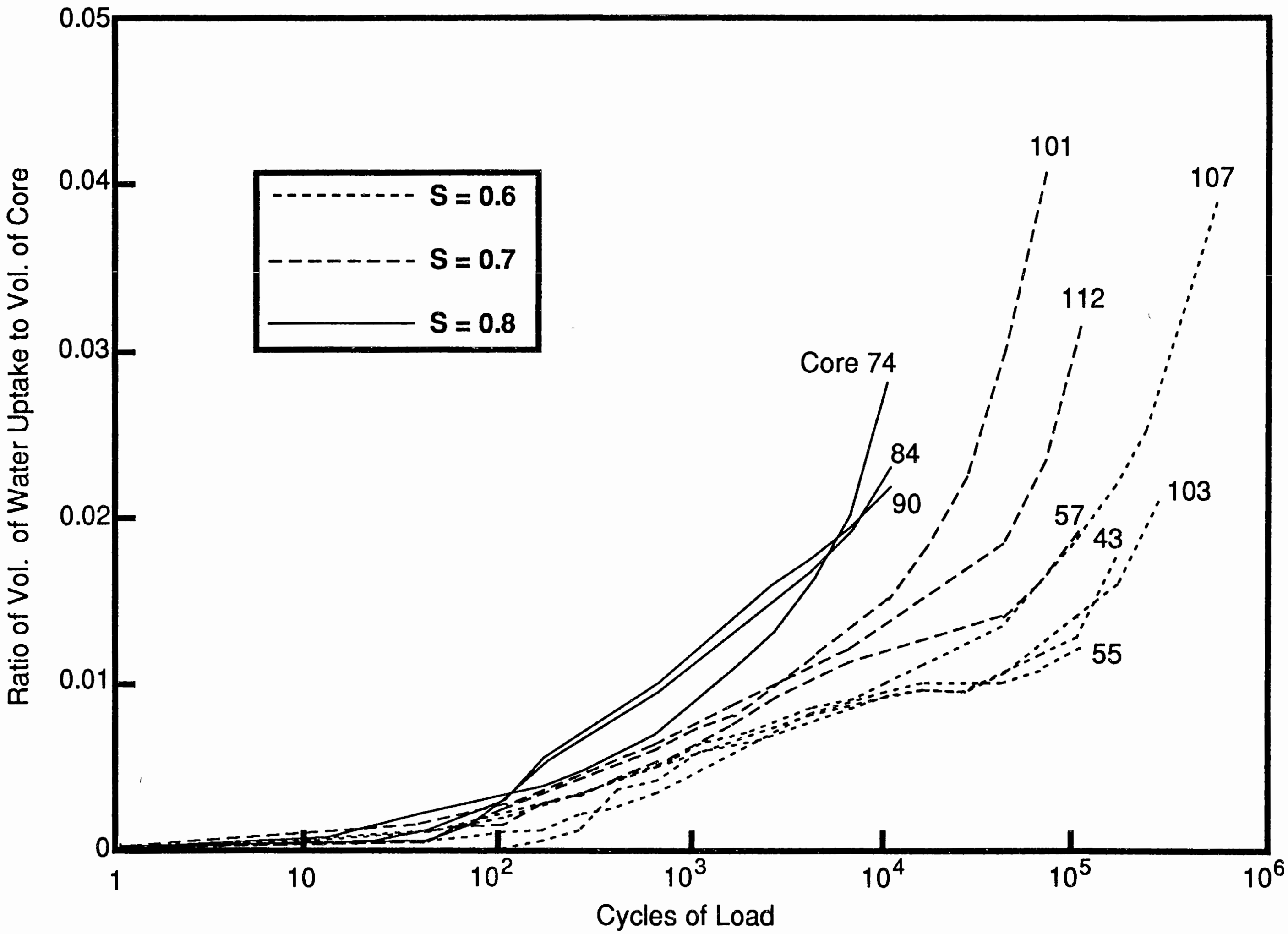


Figure 8. Water Uptake of Fatigue Tests With Confining Pressure of 7.0 MPa (Frequency of 1.0 Hz)

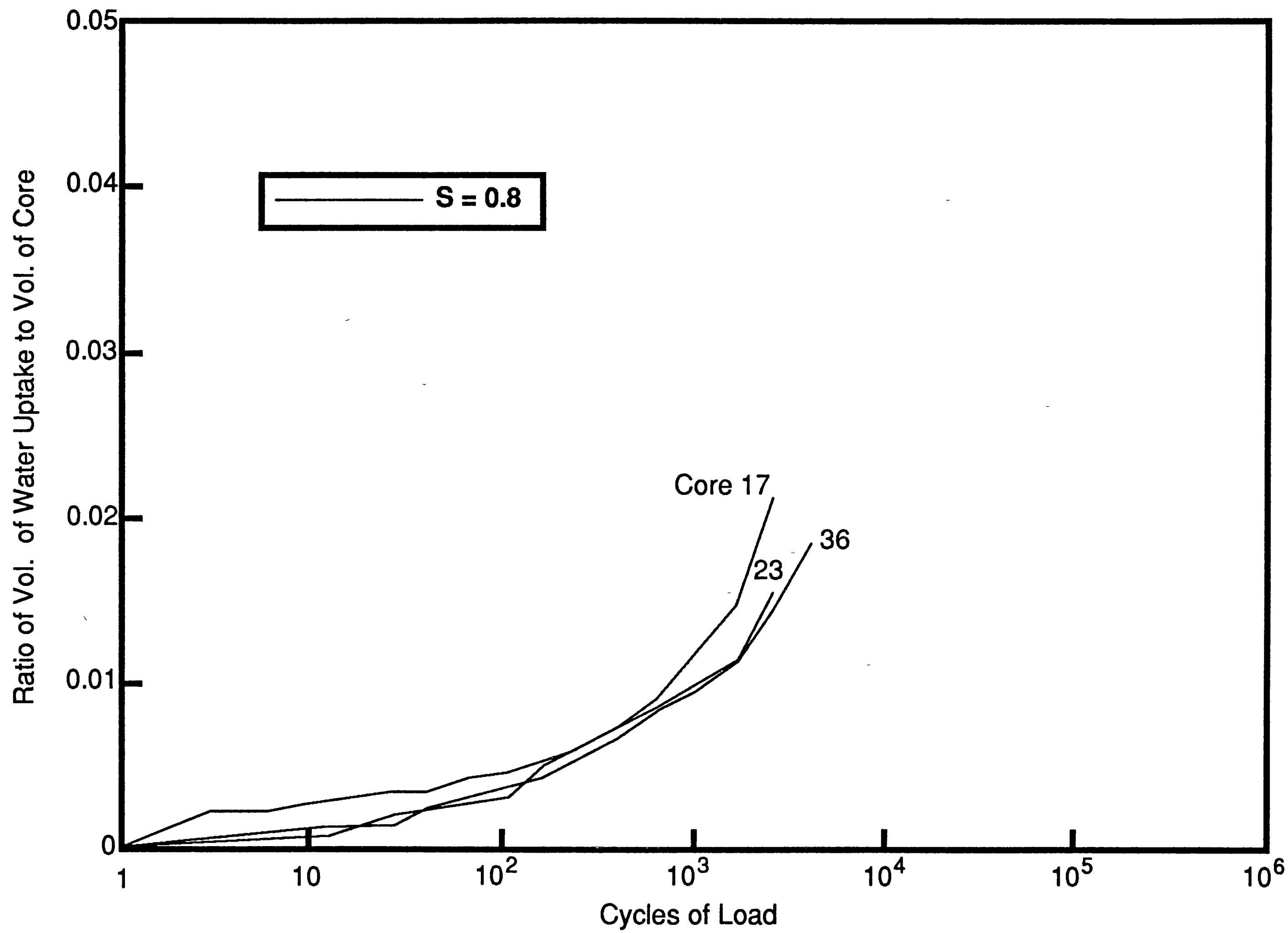


Figure 9. Water Uptake of Fatigue Tests With Confining Pressure of 3.5 MPa (Frequency of 0.1 Hz)

results from four cores tested with a maximum stress level of 70 percent are shown in Figure 10. In this plot, the axial stiffness is presented in a nondimensionalized form by dividing the stiffness of a core at various number of cycles by the initial stiffness.

Examination of cores subjected to the split cylinder test after periods of fatigue loading in a dye solution revealed that most cracks developed parallel to the direction of loading. The core with an uptake of 2 $in.^3$ and the core with an uptake of 6 $in.^3$ had similar stain patterns; in particular there was no evidence that damage extended from the surface to the interior as damage increased. There was a slight trend for cracks to be longer and more interconnected for the core with the larger uptake of dye solution.

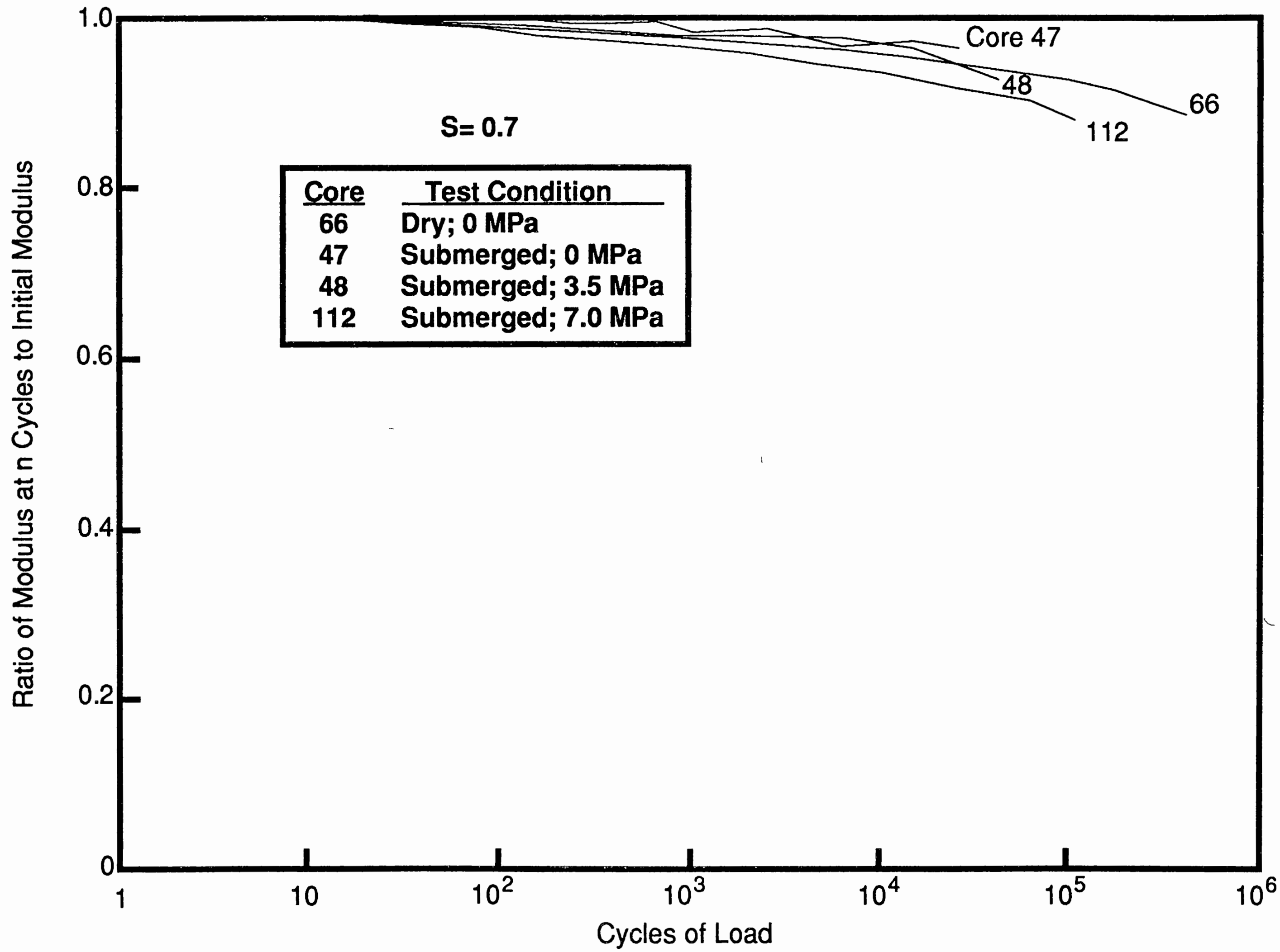


Figure 10. Reduction in Axial Stiffness Produced by Fatigue Loading

CHAPTER V

ANALYSIS AND DISCUSSION OF RESULTS

5.1 Static Tests

The rate of loading had a significant influence on the static compressive strength. In Figure 11, relative strength is plotted against rate of load on a log scale. In this figure, relative strength is the ratio of static strength of a given specimen to mean strength of cores tested at a load rate of 250 kPa/s at the same confining-moisture state. The results are in close agreement with those presented by Neville [1] (Figure 1).

Based on a review of literature and exploratory tests conducted at the beginning of this program, it is difficult to achieve saturation for high strength LWA concrete. The short 24-hour soak period employed with specimens tested under a confining pressure of 3.5 or 7.0 MPa produced only superficial saturation of zones--mostly coarse aggregate particles--at the boundary of specimens. Consequently, significant positive pore pressure was not generated by the application of load.

At the initiation of static tests, the impermeable nature of the concrete would cause the boundary of specimens to act as a watertight membrane. Eventually the level of loading in the tests was sufficient to induce fracture. Water flowed into the interior of specimens along cracks intercepting the surface; such cracks would partially defeat the confinement provided by the impermeability of the concrete. In Figure 5, the ratio of volume of water entering the triaxial cell to the volume of the specimen and the volumetric strain for the specimen are presented. At the start of the test, volumetric ratio and volumetric strain are in close agreement. When the level of load is sufficient to induce fracture, the curves for volumetric strain and the volumetric ratio diverge.

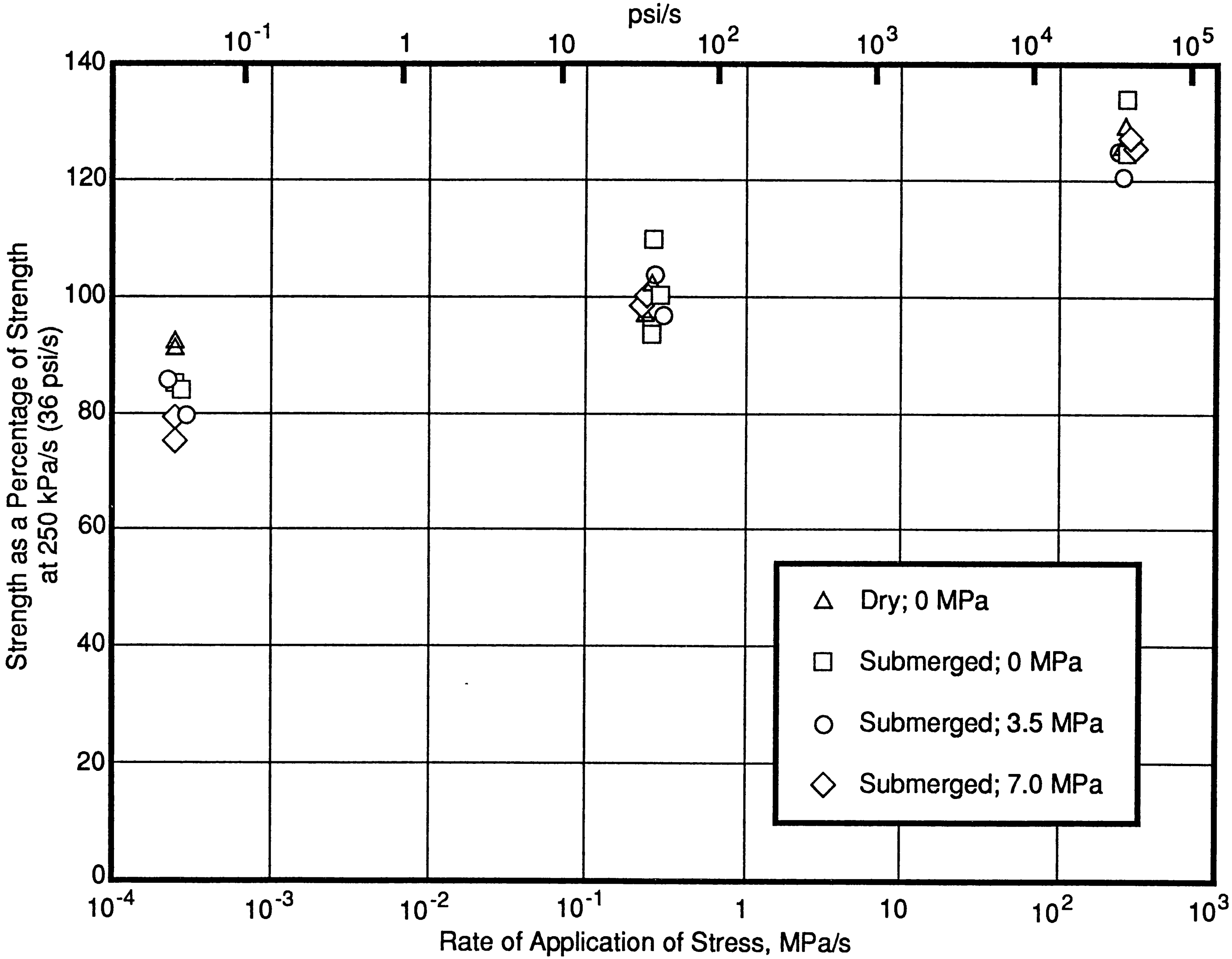


Figure 11. Influence of Load Rate on Relative Compressive Strength

Much of the attention given to the role of saturation and pore pressure has been from the perspective that concrete is a homogeneous quasi-isotropic material. However, fracture occurs at localized zones. The fact that water is able to travel along cracks and reach zones where fracture is in progress suggests that the partial confinement provided by surrounding water will be time-dependent.

The influence of confining pressure should decrease as water has greater opportunity to reach zones where fracture is occurring. Thus the influence of confining pressure should be dependent on rate of loading. If loads are applied at a slow rate, water will have more opportunity to reach an approximate equilibrium state, thereby decreasing the influence of confinement.

This hypothesis is supported by data shown in Figure 12. These results show that the benefits from confinement are not proportional to the hydrostatic pressure. In particular, the specimens tested at the lowest rate of loading exhibited almost no strength improvement when the confining pressure was increased from 3.5 to 7.0 MPa. Specimens tested at the medium or fast rates of loading exhibited almost identical increases in strength when the pressure was increased from 0 to 3.5 MPa; a smaller increase was found for specimens tested at the slow rate of loading. The results obtained here show slightly less benefit from confining pressure than was obtained by Waagaard et al. [13] but more benefit than was measured by Bjerkli [12].

5.2 Fatigue Tests

The results of the fatigue testing are presented in the form of an S-N diagram in Figure 13. In this figure, stress levels are referenced to the average static strength of four cores loaded at a rate of 250 kPa/s while in a water environment at atmospheric pressure conditions equivalent to a standard compression test. The minimum stress was 5 percent of the maximum stress level. The lines drawn through the points in Figure 13 represent a least square linear regression.

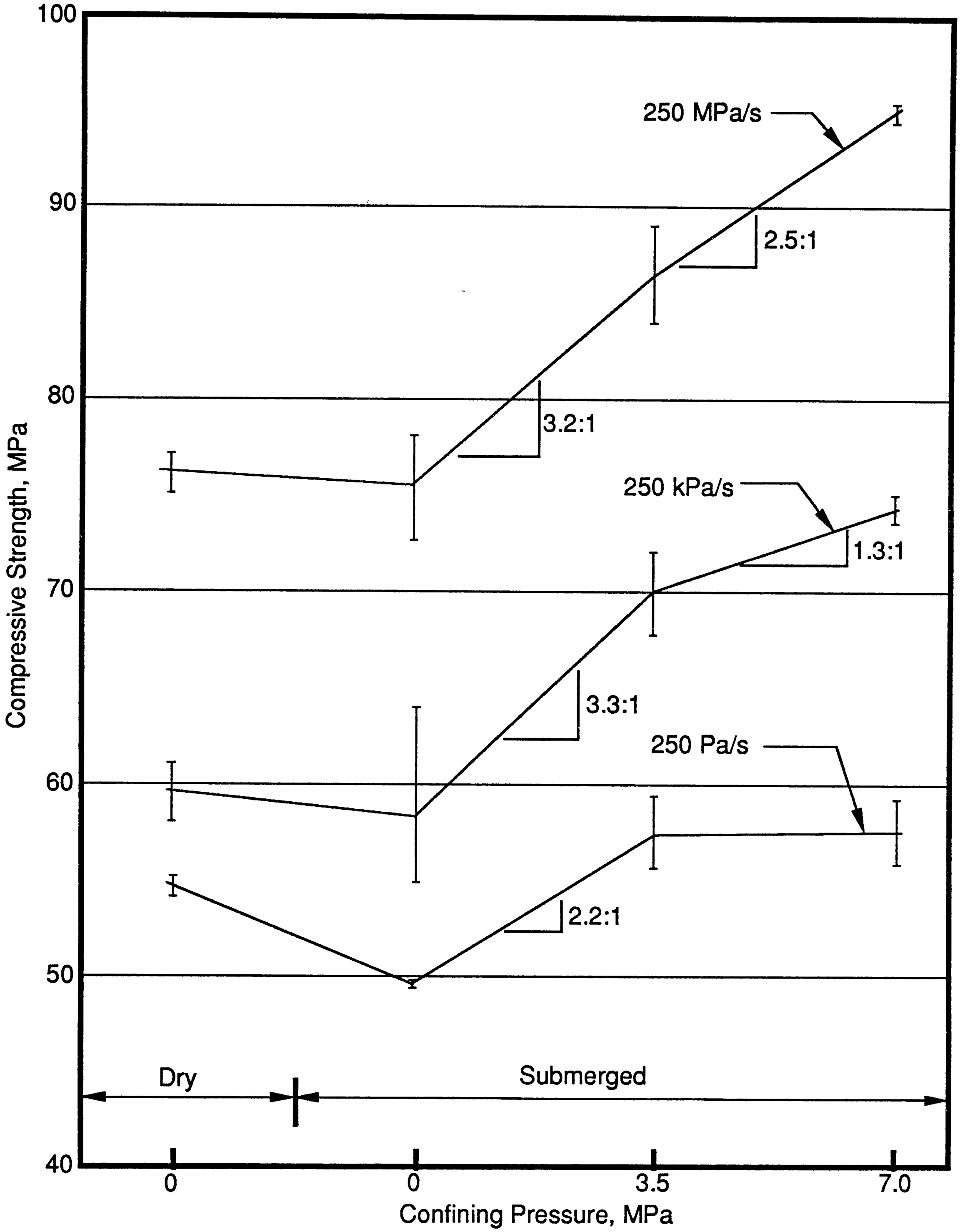


Figure 12. Influence of Moisture, Confining Pressure, and Rate of Load on Static Strength

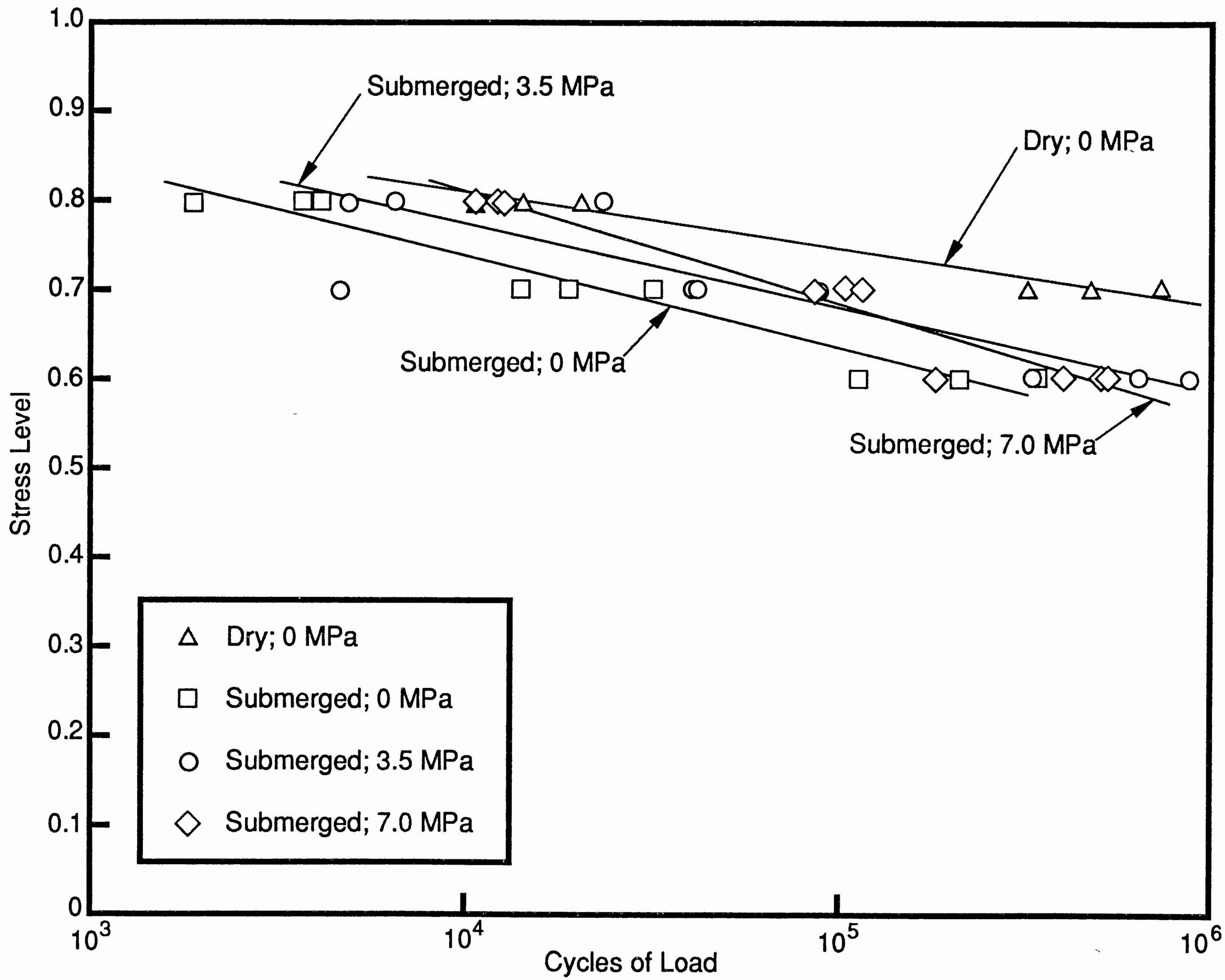


Figure 13. S-N Diagrams With Stress Level Referenced to Standard Compressive Strength

It can be seen from the figure that specimens tested in a dry condition had the longest fatigue lives while specimens tested in a water environment at zero pressure exhibited the shortest fatigue lives. Specimens tested with confining pressures of 3.5 and 7.0 MPa exhibited similar fatigue lives which were between the dry and zero pressure conditions. At a stress level of 0.8, there is a definite trend for confining pressure to benefit the fatigue life. However, for the stress level of 0.6, the benefit from confinement was significantly reduced. It can also be seen from the figure that the predicted fatigue life at a stress level of 0.6 is slightly lower with a confining pressure of 7.0 MPa than with a confining pressure of 3.5 MPa.

In Figure 14, the data are presented using stress levels referenced to the average static strength obtained at a load rate of 250 kPa/s and at the pressure-moisture condition as used in the particular fatigue tests. From Figure 14, it is evident that dry cores exhibit the greatest fatigue strength. In addition, the fatigue strength of specimens tested in water decreases with an increase in confining pressure. This suggests that benefits to static strength provided by confining pressure are temporary and should not be used to estimate fatigue capacity.

Figures 6 through 9 give the ratio of the volume of water entering the triaxial cell to the volume of sample. From these figures it can be seen that detectable quantities of water entered the triaxial cell early in the tests of specimens under confinement of 3.5 and 7.0 MPa. For tests at zero confining pressure, water was usually but not always detected entering the cell just before failure. During fatigue tests, data were sampled approximately five times per decade. Consequently the maximum water uptake shown in Figures 6 through 9 omit water entering the triaxial cell between the time of the last measurement and the time of failure. Therefore, the last points shown on these figures should not be viewed as a reliable estimate of the total water uptake at the time of failure.

Three specimens were tested at a confining pressure of 3.5 MPa, a maximum stress level of 80 percent, and a frequency of 0.1 Hz. These specimens exhibited

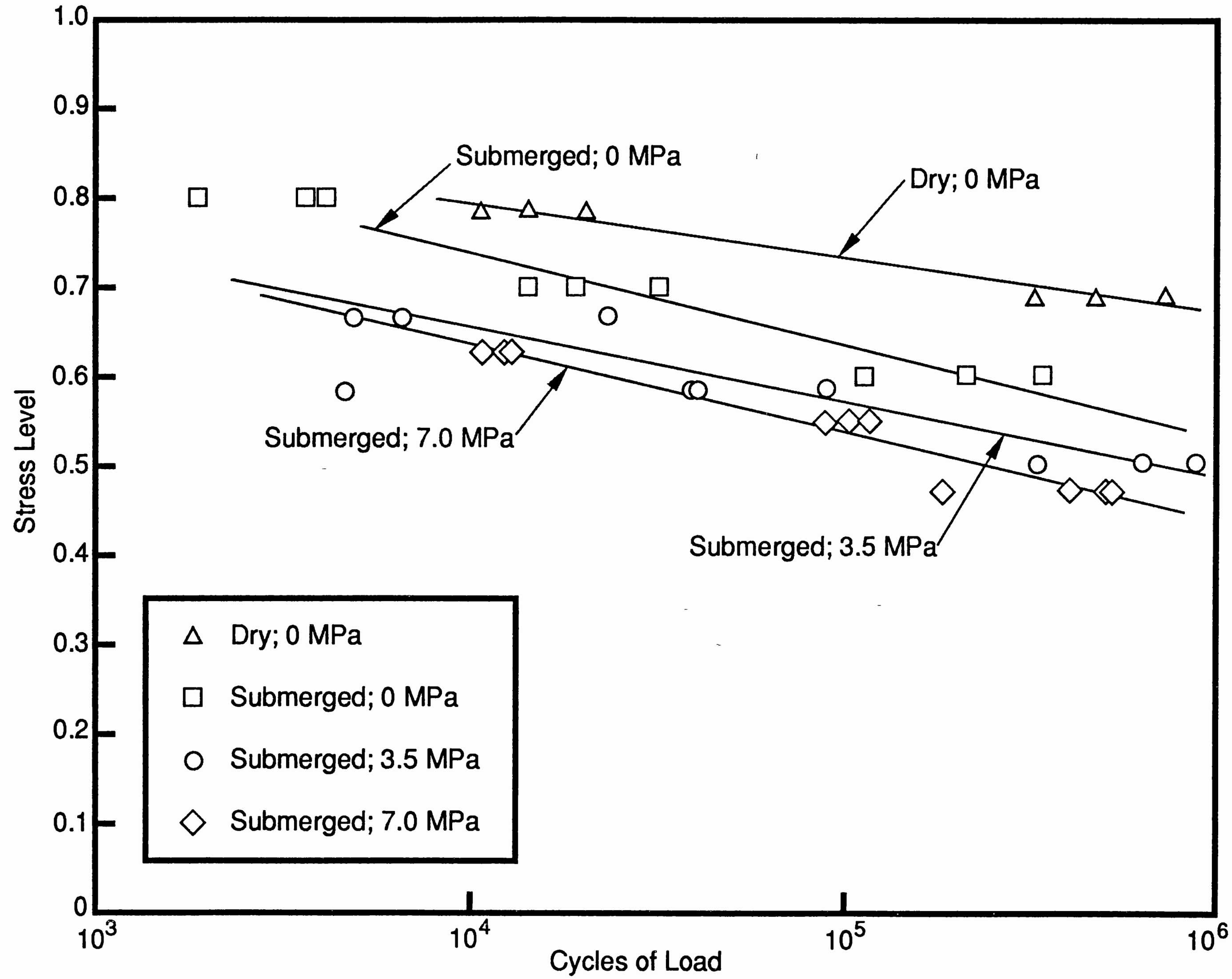
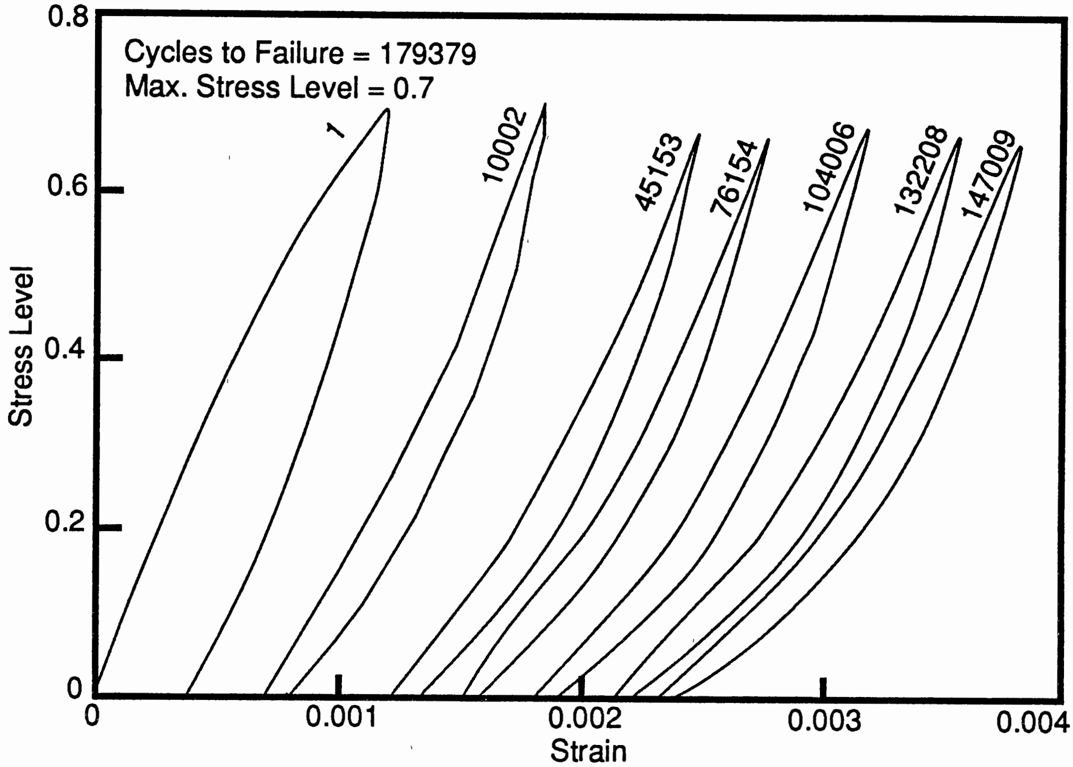


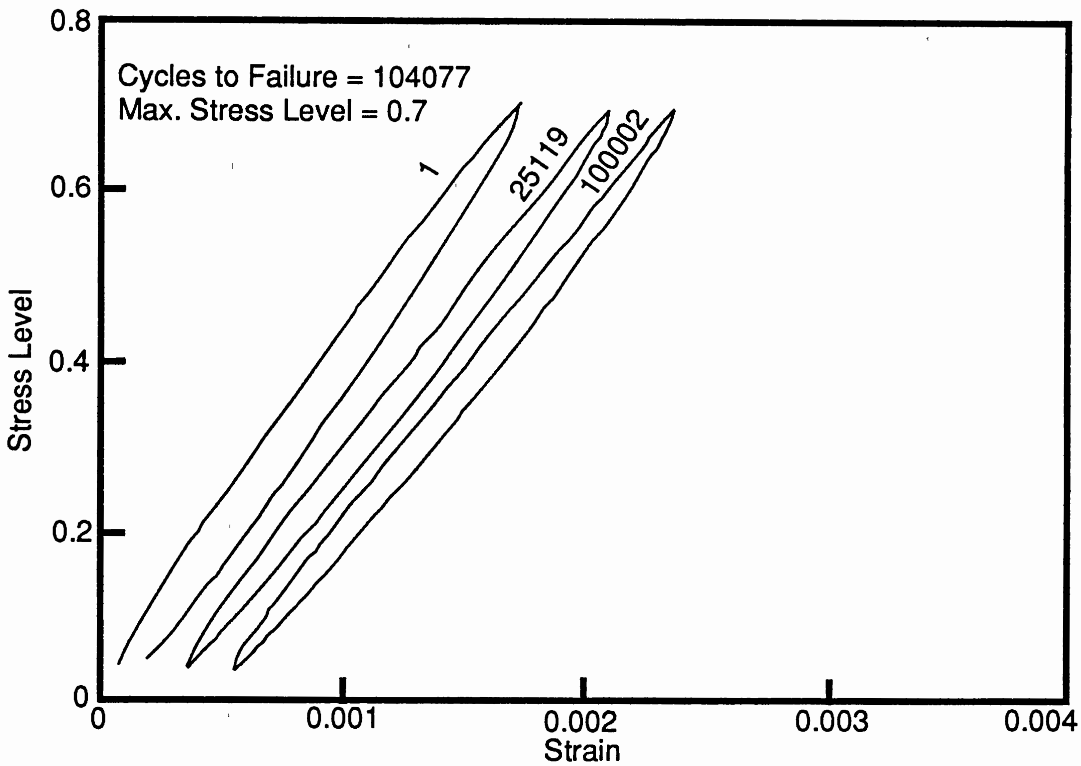
Figure 14. S-N Diagrams With Stress Level Referenced to Strengths Obtained Under Various Confinements

shorter lives than specimens tested at the same stress level and confining pressure but at a frequency of 1.0 Hz. The arithmetic mean life of specimens tested at 0.1 Hz was approximately 32 percent of the mean life for specimens tested at a frequency of 1.0 Hz.

Figure 10 shows that the axial stiffness of cylinders decreased during fatigue tests. Several researchers have presented figures showing the degradation in stress-strain properties which take place during the fatigue tests of ND concrete; Figure 15 presented by Jinawath [24] is typical. Figure 15 also presents equivalent data for the cores tested in this study. It is obvious that comparably little "drift" occurs in the stress-strain diagrams with increasing cycles for the LWA concrete. In addition, the shape of the stress-strain diagrams for LWA concrete remains almost unchanged during a test. This trend may be related to the importance of bond deterioration in the fatigue life of ND concrete.



(a) Normal Density Concrete Dry in Air [24]



(b) LWA Concrete Submerged in Water at 7.0 MPa

Figure 15. Effect of Repeated Load on Concrete Stress-Strain Diagram

CHAPTER VI

SUMMARY AND CONCLUSIONS

6.1 Summary and Recommendations

The purpose of this study was to determine the influence of hydrostatic pressure on static and fatigue strength of high strength LWA concrete. Static tests were performed using load rates of 250 Pa/s, 250 kPa/s, and 250 MPa/s. Unjacketed specimens were tested air-dried at atmospheric pressure or submerged in water with confining pressures of 0 MPa, 3.5 MPa, or 7.0 MPa. Fatigue tests were performed using the same triaxial states employed in the static tests; a maximum stress level of 60, 70, or 80 percent of the compressive strength; and a load frequency of 0.1 or 1.0 Hz. The minimum stress level was 5 percent of the respective maximum stress.

The results of this study can be summarized as follows:

1. Influence of load rate on high strength LWA concrete is similar to the influence that has been reported for normal density concrete.
2. The benefit of confining pressure on static strength was more pronounced for higher rates of load. For a low rate of load (250 Pa/sec), increasing the confining pressure from 3.5 to 7.0 MPa did not result in an increase in static strength.
3. Fatigue life of concrete tested in a submerged condition was significantly less than for concrete tested in a dry condition.
4. The fatigue strength of concrete tested in a submerged condition with a confining pressure of 3.5 or 7.0 MPa was similar and slightly greater than the strength of concrete without confining pressure.

5. As reported by others, the fatigue strength of concrete is higher when tested at a higher frequency.

6.2 Suggestions for Future Work

If the high strength LWA concrete in this study had been subjected to a conventional permeability test, it would have been found to be extremely impermeable. However, when cracks formed under load, water was able to penetrate the concrete in significant quantities.

Since the static tests were conducted under monotonically increasing load, equilibrium of water flow and pressure would have been difficult to achieve in the presence of active crack propagation. In fatigue tests, the movement of water into cracks would be influenced by the pulsating width of cracks caused by cycled loading. Researchers have questioned whether "wedge action" of water in cracks can influence fatigue behavior.

It is suggested that tests be conducted which would emphasize the influence of positive pore pressure in zones at the tip of cracks. For example, fatigue tests would be conducted with interspersed rest periods. The rest periods would allow water pressure in the crack zone to approach equilibrium with the surrounding pressure. Further research on the influence of pore pressure is also needed for ND and LWA concrete of conventional strength.

REFERENCES

[1] Neville, A.M. *Hardened Concrete: Physical and Mechanical Aspects.* ACI Monograph, No. 6, 1971.

[2] Neville, A. M. *Properties of Concrete.* 3rd Ed. London: Pitman Publishing, 1981.

[3] Butler, J.E. "The Influence of Pore Pressure Upon Concrete." *Magazine of Concrete Research,* Vol. 33, No. 114 (Mar. 1981), pp. 3-17.

[4] Haynes, H.H. "Permeability of Concrete in Sea Water." *Performance of Concrete in Marine Environment.* ACI SP 65-2, 1980, pp. 21-38.

[5] Buenfeld, N.R., and Newman, J.B. "The Permeability of Concrete in a Marine Environment." *Magazine of Concrete Research*, Vol. 36, No. 127 (June 1984), pp. 67-80.

[6] Richart, F.E., Brandtzaeg, A.,and Brown, R.L. "A Study of the Failure of Concrete Under Combined Compressive Stress." Bulletin No. 185. Engineering Experiment Station, University of Illinois, 1982.

[7] Hansen, J.A. "Strength of Structural Lightweight Concrete Under Combined Stress." *Journal of the Research and Development Laboratories, PCA*, Vol. 5, No. 1 (Jan. 1963), pp. 39-46.

[8] Terzaghi, K. "Stress Conditions for the Failure of Saturated Concrete and Rock." *ASTM Proceedings*, Vol. 45 (1945), pp. 777-801.

[9] Haynes, H.H., and Highberg, R.S. "Concrete Properties At Ocean Depths." *Journal of the Waterways Harbors and Coastal Engineering Division, ASCE* (Nov. 1976), pp. 455-470.

[10] Haynes, H.H., and Underbakke, L.D. "Compressive Strength of Freshly Mixed Concrete Placed, Cured, and Tested in the Deep Ocean." Technical Note N-1603. Civil Engineering Laboratory, Naval Construction Battalion Center, Port Hueneme, CA, 1981.

[11] Clayton, N. "Concrete Strength Loss From Water Pressurization." *International Conference on Concrete in the Marine Environment.* The Concrete Society, London, England, Sept. 1986, pp. 177-186.

[12] Bjerkeli, L.M. "Water Pressure on Concrete Structures," Ph.D. Thesis, University of Trondheim, Norway, 1990.

[13] Waagaard, K., Kepp, B., and Stemland, H. "Fatigue of High Strength Lightweight Aggregate Concrete." *Symposium on Utilization of High Strength Concrete*, Stavanger, Norway, 1987, pp. 291-306.

[14] Norby, G.M. "Fatigue of Concrete—A Review of Research." *Journal of the ACI* (Aug. 1958), pp. 191-220.

[15] Murdock, J.W. "A Critical Review of Research on Fatigue of Plain Concrete," Bulletin No. 475. Engineering Experiment Station, University of Illinois, Urbana, IL, 1965, pp. 1-25.

[16] Viswanathan, R. "Pore Pressure Effects on the Mechanical Properties of Concrete." Ph.D dissertation, Oklahoma State University, 1982.

[17] Leeuwen, J., and Siemes, A.J.M. "Fatigue of Concrete, Part 2." Memo-78-80-by-SIM/PEM dd 78.06.22. Institute TNO Voor Bouwmaterialen en Bouwconstructies, Holland, 1978.

[18] Waagaard, K. "Fatigue Strength Evaluation of Offshore Concrete Structures." *American Concrete Institute*, SP 75-18, 1982, pp. 373-397.

[19] Siemes, A.J.M. "Fatigue Evaluation of Concrete Structures: Fatigue Strength Procedure for Concrete Structures." *Heron*, Vol. 33, No. 3 (1988), pp. 40-60.

[20] Siemes, A.J.M. "Exploratory Investigation Into the Mechanical Properties of Three New Types of Concrete." *TNO-IBBC*, Report B-89-1989.

[21] Muguruma, H., Watanabe, F., and Kontani, O. "Low-Cycle Fatigue Behaviours of Plain Concrete Under Submerged Condition." *Transactions of the Japan Concrete Institute,* Vol. 6 (1984), pp. 215-222.

[22] Muguruma, H. and Watanabe, F. "Low-Cycle Fatigue Behavior of Concrete and AE Monitoring Under Submerged Condition." *Proceedings of the 30th Japan Congress on Material Research.* The Society of Materials Science, Kyoto, Japan, 1986, pp. 151-158.

[23] Petkovic, G.,Lenschow, R., Stemland, H., and Rosseland, S. "Fatigue of High-Strength Concrete." *American Concrete Institute*, SP 121-25, 1990, pp. 505-525.

[24] Jinawath, P. "Cumulative Fatigue Damage of Plain Concrete in Compression." Ph.D. thesis, University of Leeds, London, England, 1974.

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