CREEP CHARACTERISTICS OF A LIGHT-WEIGHT AGGREGATE CONCRETE

by

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TABLE OF CONTENTS

TABLE OF CONTENTS - Continued

CREEP CHARACTERISTICS OF A LIGHT-WEIGHT AGGREGATE CONCRETE

CHAPTER I

INTRODUCTION

First Evidences of Creep

Creep, or time yield, of concrete was first observed by I. R. Woolson in 1905. Woolson cast fine aggregate concrete in thin metal tubes, and exerted high compressive loads upon the longitudinal axes of these tubes until severe deformation resulted. He cut the tubes open, expecting to find the encased concrete crumpled and broken. To his surprise, the concrete had remained whole, even though undergoing excessive permanent deformation. Woolson necessarily concluded that the concrete had flowed plastically (32, p. 459).

Investigators soon began to notice continued deformation of concrete components in service. This phenomenom was alternately termed "creep", "plastic flow", or "time yield" to distinguish it from the phenomenom of elastic deformation.

Based on his work with large reinforced concrete beams, F. R. McMillan proposed that time yield consisted of two parts: shrinkage strains, and plastic volume change. He was also the first to observe that rate of yielding was approximately proportional to magnitude of applied stress (19, p. 251-252).

Almon H. Fuller and Charles C. Moore, working with medium strength stone concrete cylinders, corroborated McMillan's findings (12, p. 302-310).

Earl B. Smith, experimenting with reinforced concrete beams and gravel concrete cylinders, reported that instantaneous recovery of deformation upon unloading was practically equal to initial elastic deformation. Some slight plastic recovery with time was noted. Smith was the first investigator to use unloaded control specimens in an attempt to divorce the effects of shrinkage from the effects of plastic flow (28, p. 317-321).

Chapter II of this thesis is devoted to a further discussion of creep; its general nature, factors affecting, and theories of its mechanism.

Creep Affects Columns, Beams, Slabs

The effect of creep in concrete is to relieve points of high stress concentration and redistribute stress more equitably through the mechanism of superelastic strain. Axially loaded columns will tend to "flow-out" from beneath their loads, transferring stress from the concrete to the longitudinal reinforcing steel (6, p. 385).

Similarly, creep allows the neutral axis of a beam, loaded in flexure, to approach the reinforcing steel. This movement of neutral axis provides increased concrete cross-section to take compressive stress. The tensile stress carried by the reinforcing steel is increased.

Creep effect in a slab is fundamentally the same as its effect in a beam.

Creep Affects Loss of Prestress

A concrete beam is prestressed by imposing a compressive stress distribution on the beam, such that it cancels all tensile stresses that may be imposed on the concrete by service conditions. In general, this is accomplished by loading the beam, eccentric to its longitudinal axis, by means of internally placed steel tendons.

Prestress may be accomplished either by post-tensioning or by pre-tensioning. In post-tensioning systems, the steel tendons are tensioned after the hardening of the concrete beam, and the tendon load is transmitted to the concrete through the action of end anchors.

In pre-tensioning, the tendons are tensioned before the beam is cast. Upon hardening of the beam, the tendon ends are released. The tendon load enters the concrete through steel-concrete bond over the length of the tendons.

Any shortening of the steel tendons after initial tensioning will result in a loss of prestress. Concrete shrinkage and plastic flow both allow shortening of the tendons. Elastic strain in the concrete due directly to the prestress will also allow steel shortening.

For further information on prestress and prestress losses, the reader is referred to excellent works by Gustave Magnel (20) and by T. Y. Lin (14, p. 52-109).

This thesis is primarily concerned with loss of prestress as a result of creep in Lite Rock concrete.

"Lite Rock" Concrete

"Lite Rock" is the trade name of an expanded shale aggregate and the concretes made thereof; products of the Empire Building Material Company of Portland, Oregon.

The virgin shale is quarried near Banks, Oregon. The shale is first crushed and sorted to the desired sizes and gradations, and then placed in a large rotary kiln. The kiln is heated to 2200° F. at which temperature the shale begins to melt. Gases are evolved which expand the now viscous shale, and develop within it a cellular structure. The outer surface of each shale particle is molten, and upon cooling forms a glaze-like coating over the cellular interior (Fig. 1).

In 1951, Dwight D. Ritchie, a graduate student in Civil Engineering at Oregon State College, conducted an extensive investigation into the properties of Lite Rock concrete. His conclusions as to its properties are listed in his Master's thesis. The more important conclusions are listed below[1] (25, p. 74).

> The following conclusions are drawn concerning Lite Rock concrete:
>
> 1. Unit weight, dry, is from 60 to 80 pounds per cubic foot.
>
> 4. The compressive strength ranges from 1200 to 4200 pounds per square inch, depending upon the cement factor and the maximum size of aggregate.
>
> 5. Less strength is gained beyond the seven-day curing period than with heavier concrete.

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[1] Since the date of these tests, compressive strengths have been realized up to nearly 7000 pounds per square inch.

6. Resistance to bond and shear is in accordance with compressive strength.

7. Absorption is not excessive when considered on volume basis.

8. Twenty-eight day shrinkage is less than that for gravel concrete.

The above will serve as a brief but adequate description of Lite Rock concrete. Figure 2 depicts a cross-section of a standard 6" x 12" test cylinder made with Lite Rock (Fig. 2).

Need for Information on Creep of "Lite Rock"

Knowledge of the creep characteristics of any concrete is to be desired; as creep phenomenom effects the deformation and stress distribution of all concrete structural members. This information becomes especially important in reference to concrete prestressing, as previously detailed in this chapter.

In the past, Lite Rock concrete has been successfully used in building blocks, reinforced concrete columns, and reinforced concrete beams. Recently, the Empire Building Materials Company has begun to fabricate prestressed beams of Lite Rock. The first of these were post-tensioned, but more recently the company has constructed large tendon tensioning beds, and plans to begin pre-tensioning. As of this date the company is pre-tensioning conventional concrete beams.

In order to adequately allow for prestress losses, the company requires accurate information on the extent and character of creep in Lite Rock. The Bureau of Public Roads Manual Criteria for Prestressed Bridges gives the loss of prestress that may be expected for normal stone concrete, but gives no like information on other concretes. It states that such information of losses from the various causes shall be arrived at through tests (30, p. 1 and p. 13). It was for cooperation in the performance of such tests on Lite Rock that Frank Spangler, President of the Empire Building Materials Company, contacted Oregon State College.

The tests outlined in this thesis are the first in a series aimed at discerning some of the necessary information for calculation prestress losses in concrete beams of Lite Rock expanded shale aggregate.

General Outline and Scope of Research

Prismoidal specimens of plain Lite Rock concrete were loaded under conditions analogous to those experienced by beams loaded only by concentric stressing tendons. This is accomplished by two methods differing mechanically and in instrumentation. The tests divide themselves into three groupings, as follows:

I. Six specimens each, of two different mixes of Lite Rock, were loaded axially in compression by steel yokes at either end. The yokes were pulled together by two steel bars in tension. Variation of strain in the steel bars was measured by SR-4 electric strain gages affixed to the bars. Electric strain gages were also placed on the prisms in an attempt to measure concrete strain directly (Fig. 3). Initial loads in the concrete varied from 20% to 80% of ultimate strength. Stress was maintained and recordings continued for some 4000 hours.

II. Three specimens each, of Lite Rock concrete, and normal stone concrete, were loaded concurrently under the loading units described above (Fig. 4). Levels of loading were essentially the same in all six units. One of the Lite Rock specimens had electric strain gages placed about its girth. These gages were used in hopes of obtaining information on Poisson's effect (Fig. 5). This loading continues, with over 1500 hours to date.

III. Three specimens, one each of two different mixes of Lite Rock, and one of Lite Rock coarse aggregate and river sand fine aggregate, were loaded axially in compression by an Olsen 30,000# testing machine. Load in the concrete was directly obtainable from the machine's load dial and balance arm (Figs. 7 and 8).

In each case, load fall-off was recorded as a function of time under load. In all, five different mixes were involved in the program. All were of very high strength, with ultimate stresses ranging from 4900 to nearly 7000 pounds per square inch. Three of the mixes consisted almost entirely of Lite Rock aggregate, with very small amounts of natural sand. One mix utilized natural sand exclusively for fine aggregate and Lite Rock for coarse aggregate. The fifth mix used no Lite Rock, but was composed of natural stone and sand aggregates.

CHAPTER II

THEORY

Variables Affecting Creep

Woolson (32), Smith (28), McMillan (18, 19), Fuller and Moore (12) were among the pioneer investigators of creep (or as they termed it: "time yield") in concrete. Even in their cursory investigations, they noted that creep seemed to be affected by a great many variables. In his 1921 report of his investigations, F. R. McMillan reported the effect on "time yield" of varying water content, concrete density, mix proportions, and storage moisture conditions (18, p. 161).

In 1925, Raymond E. Davis and his assistants at the University of California, began an extensive series of investigations of creep in concrete; investigations which have been continued to date. A large share of our knowledge concerning the behavior of concrete under constant load--the effect of reinforcing, curing conditions, and a multitude of other factors--we owe to Davis and his work at the University of California (3, 4, 5, 6, 7, and 8).

The list below details what is known concerning the effects of the many variables upon character and magnitude of concrete creep. It is believed that this listing will aid in the understanding of the discussion of the theories of creep which appears later in this chapter.

After over thirty years of continuous testing of concrete columns, test cylinders, and beams; under constant load and varying conditions; Davis and others have reached the following conclusions:

> 1. Plastic flow (creep: Davis) occurs at a decreasing rate for at least ten years, in concrete under constant load. Flow is well over 95% complete in five years (5, p. 317).
>
> 2. Flow is of practical importance only for the first two years in plain concrete; only for the first year in dry reinforced concrete; and only for the first few months in wet reinforced concrete (6, p. 354).
>
> 3. At least at low stress levels, other factors held constant, rate of creep is a linear function of applied stress. Lorman sets the upper limit at 1500 psi. for normal stone concrete (15, p. 1084). Shank holds that a linear relationship is maintained up to stress as high as 75% of ultimate (26, p. 493). At stress approaching ultimate, creep rate appears to vary as a greater than linear function of applied stress.
>
> 4. The older the concrete at loading, the less the eventual total creep (6, p. 370).
>
> 5. Rate of creep varies with humidity. Concrete stored in air will creep at a rate roughly three times greater than that of concrete stored under water (4, p. 321).
>
> 6. Both paste content of concrete and water-cement ratio should be minimum for minimum creep and shrinkage (7, p. 390).

7. Creep varies inversely with modulus of elasticity but does not seem independently affected by ultimate strength (4, p. 321).

8. Creep is less in concretes with well graded aggregate. The larger the aggregate, the less the creep (7, p. 386).

9. Large members show less ultimate creep and shrinkage than do small ones (7, p. 389).

10. Concretes made with cements low in alkalies (soda and pottasa) and high in tri-calcium silicate exhibit less shrinkage, and probably less creep (7, p. 385).

11. The longer the cure, the less the ultimate shrinkage and creep (6, p. 383).

12. In general; any factor that will affect concrete shrinkage, will similarly affect creep.

13. Type of aggregate used in the concrete has its effect. Aggregates in order of increasing deformation are: limestone, quartz, granite, gravel, basalt, sandstone (3, p. 856).

14. Creep deformation may be several times as great as initial elastic deformation (3, p. 885).

15. When load is released, there is an immediate elastic recovery, followed by a plastic recovery at a decreasing rate. The elastic recovery is 50% to 90% of the initial elastic deformation. The plastic recovery is much less than the plastic deformation that took place under load (3, p. 871-875).

16. The magnitude of instantaneous deformation, rate of creep, magnitude of instantaneous recovery, and rate of plastic recovery, become less at each new loading (3, p. 871-875).

17. Creep under tensile stress was found to be greater at first, but less later, than creep under compressive stress (5, p. 323).

18. All factors affecting creep under compressive stress also affect creep under tensile stress, in the same way and approximately to the same extent.

The above observations have led to five different theories as to the nature of creep.

Theories of Creep

Mechanical Deformation Theory. Freyssinet attributes behavior of concrete under load to delayed elastic deformation of its capillary structure. This delay is due to water pressure in the interior; pressure which is relieved slowly by evaporation and seepage. All such deformation is eventually recoverable subsequent to unloading (11, p. 49-51).

Plastic Theory. It is postulated that cement is a plastic solid and that creep is of the nature of crystalline flow, i.e. a result of slippage along planes within the crystal lattice (22, p. 48).

Viscous Theory. Viscous theory assumes creep in concrete to be in the form of viscous flow, with movement of cement paste particles, one over the other. Flow is resisted by the aggregate, and the aggregate becomes more highly stressed as the cement paste loses stress. As rate of creep is proportional to applied stress (assumed), rate of creep decreases as stress is transferred to aggregate. Therefore, total creep depends on viscosity of cement paste, elasticity of aggregate, and plastic creep (if any) within aggregate (22, p. 49-50).

Seepage Theory. Creep in concrete is assumed due to loss of adsorbed water from cement gel.

Cement paste is at first a mixture of solutions and a colloid. Heat of hydration slowly transforms the colloid into a calcium silicate gel. As the colloid solution turns to gel, a network of solid phase permeated by liquid phase is formed. The gel at first retains an enormous amount of water which can be lost by evaporation, or squeezed out under pressure. Hydration is very slow and probably continues indefinitely.

Water is retained in the concrete mass in three ways: (1) chemically combined; (2) colloidally adsorbed; (3) free water mechanically held in gel capillaries.

White found that immersing concrete in benzene caused no volume change in the concrete (22, p. 51). As benzene cannot hydrate lime, be held as water of crystallization, or be adsorbed in the gel, Lynam inferred that free water (mechanically held) has no effect on volume change. Hence, he describes both shrinkage and creep as results of the loss of colloidally adsorbed water; although in one case it may be drawn out by evaporation, and in the other, expelled by pressure (16).

It is held that seepage creep may occur even though the concrete does not lose weight. Adsorbed water may be forced into voids as free water (22, p. 53).

Elastic After-Effect Theory. Maney contends that creep at working stresses is really an elastic strain due to uneven shrinking. He bases his contention on experimental evidence of uneven shrinkage in concrete cylinders and on the assumption that loaded specimens creep twice as much as unloaded specimens. This leads him to conclude that all recovery must be elastic; further, if we can prevent shrinkage, we will prevent creep. The well established fact that total strain recovery is never realized is laid to an increasing value of Young's modulus (21, p. 1021-1029).

McHenry also credits creep to "imperfect elasticity", and claims that creep is fully recoverable. In support of his thesis, he cites data for certain resins and volcanic rock and holds these to be similar in behavior to concrete (17, p. 1069-1071).

Leaderman, in consent with McHenry, suggests that the creep process involves the motion of long molecule chains (as in the elasticity of rubber) (13, p. 1085-1087).

Discussion of Theory

Freyssinet has theorized that elastic deformation is delayed by water pressure in the concrete interstices and that the delayed deformation may slowly take place as the water is lost through seepage and evaporation. This necessitates a loss in weight accompanying the creep process. Although such is normally the case, it has been demonstrated that creep may occur independent of moisture movement (21, p. 1022-1023). Furthermore, both this theory and McHenry's suffer from the fact that it has been conclusively shown that full recovery is not realized (3, p. 871-875).

Exponents of Plastic Flow Theory point to the work of Vogt whose experiments indicate that concrete displays the same modes of deformation under sustained load as do the metals when stressed beyond the yield point (31). However, plastic solids flow only after a finite yield point is reached, their plastic deformation (set) is not recoverable with time, and their volume is maintained constant. But Bingham and Reiner, in their classical work, The Rheological Properties of Cement and

Cement-Mortar Stone, show that cement-stone will yield under any finite stress and exhibits no yield point (2, p. 94). Davis and others have shown that concrete does recover a portion of its plastic deformation after unloading and that concrete does not maintain a constant volume under sustained load (3, p. 871-875). Nevertheless, plastic flow may be a contributing factor to creep at high stress levels. Evans has reported plastic flow in rocks at stresses as low as 1000 pounds per square inch (9, p. 145-158).

As a result of their exhaustive work, Bingham and Reiner concluded that cement-stone was a highly viscous fluid. Rate of creep being proportional to applied stress (except at stresses approaching the ultimate) indicates that cement-stone may be considered a Newtonian Fluid. The apparent paradox that concrete appears to be approaching an equilibrium at a finite strain, Bingham and Reiner explain as a transfer of stress from viscous cement paste to elastic aggregate. Equilibrium is reached when all particles of aggregate are touching, acting as a stress bridge over the paste. Time yield, therefore, is partially an elastic effect, for as stress is transferred to the aggregate particles, they experience elastic deformation (2, p. 95).

Viscous flow implies constant volume, but concrete creep is normally associated with a volume change. Therefore, viscous flow cannot be totality of creep.

The first proponent of the Seepage Theory was O. Faber, who, in a paper given before the Institution of Civil Engineers in 1927, said, "The colloidal conception of the constitution of concrete, and the known properties of inorganic colloids, adequately account for all the observed properties in connection with shrinkage, hardening, and plastic yielding of concrete" (10, p. 662). After nine years of experimental research, Davis also proposed that the majority of flow is due to seepage of colloidal water from the cement gel (6, p. 355-357, and 7, p. 384).

Moisture movement has been observed in aggregates. Movement in limestone was determined to be about 1/20 that in cement paste, and movement in sandstone about 1/3 (22, p. 52). That less creep be observed in a limestone concrete than in a sandstone concrete would be in agreement with Seepage Theory. Davis found that this was the case (3, p. 856, and 7, p. 387).

The most serious objection to Seepage Theory arises from the observance of tensile creep. In light of Seepage Theory, conditions of high humidity would produce a pressure gradient favorable to tensile creep. Yet, tensile creep of concrete immersed in water has been found to be only one-tenth that of concrete stored in dry air (5, p. 323-324).

Maney's stand that apparent time yield is an elastic effect due to non-uniform shrinkage has been seriously challenged by Pickett. According to Maney, the elastic effect of non-uniform shrinkage and the elastic effect of load, interact to yield a greater deformation than would be obtained by summing their individual effects (21, p. 1024). Pickett points out that, "For elastic materials this principle (of superposition) requires that shrinkage effects and load effects be entirely independent, which means that shrinkage and load could not interact to cause additional deformation. Therefore, the effect of shrinkage upon creep cannot be explained as elastic deformation" (23, p. 1033). Maney, in closure, denies validity of the principle of superposition in this instance on the grounds that the concrete is deformed and changes shape (21, p. 1036). The writer is of the opinion that the deformation is insufficient (approximately 0.01%) to invalidate the principle of superposition.

The writer proposes that creep at stresses not exceeding 70% to 80% of ultimate is probably due to the combined effects of viscous flow and seepage of colloidally adsorbed water. Creep and shrinkage, therefore, are not distinguishable in concrete under load. At very high stress intercrystalline slip in aggregate may become significant.

CHAPTER III

TESTING PROCEDURES

Specimens - Description

Forty-two 2" x 2" x 18" concrete prisms of five different mixes, were cast as specimens by the Empire Building Materials Company of Portland, Oregon. The specimens were cast in plywood forms constructed by the writer.

The first set of nine specimens was cast February 22, 1957. The mix used for these specimens was designated "A". The forms were stripped the following day, and two thumbtacks were pressed into each of two opposite sides of each specimen. These thumbtacks were placed at 10" centers to serve as gage points for a 10" Whittemore Extensometer. The nine specimens remained under open shed cover until February 25th, when they were removed for steam curing. Temperatures in the shed remained in the 40's during the term of storage.

In the steam room, the temperature was raised to 160° F. over a period of four hours. This temperature was maintained for twelve hours and then dropped to 140° F. during the next eight hours. At the end of this time, the specimens were removed to open storage, where they remained until delivered to the Civil Engineering Laboratories at Oregon State College on March 7th.

Two 6" x 12" standard test cylinders were cast and cured with the prisms.

On March 1st, twelve more specimens were cast of a mix, "B". They were stripped on the following day, and thumbtacks were imbedded as before. After four days of open storage, the specimens were steamed in the same manner as detailed for the "A" specimens. Upon removal from the steam rooms, they returned to open storage until delivery, March 7th.

Two 6" x 12" test cylinders were cast and cured with these prisms.

These first two mixes, "A" and "B", contained both coarse and fine Lite Rock aggregate, with small amounts of river sand fine aggregate in addition.

Later, in June of the same year, Mr. Spangler had cast, cured, and delivered, six more specimens. These were of a mix, "M". The "M" specimens were not steam cured. While the coarse aggregate in mix "M" was again Lite Rock, the fine aggregate in its entirety was natural river sand.

The "M" specimens were accompanied by six 2" x 2" test prisms of lengths varying from 2" to 4".

Six "L" and nine "S" specimens were cast by

Mr. Spangler on December 20, and 23, 1957, respectively. The "L" mix is an ultra-high strength mix, using Lite Rock aggregate similar to mixes "A" and "B". The "S" mix is a natural stone concrete of strength comparable to mix "L". Both of these mixes underwent curing as outlined for mix "A".

These latest specimens were accompanied by six 6" x 12" test cylinders, three of each mix.

All specimens in this program were cast from batch mixes prepared for larger structural elements. The smal. prisms were cured in conjunction with the large members.

Table No. 1 gives a description of each of the five mixes used in the test program. Table No. 2 describes the physical characteristics of the five resultant concretes.

TABLE NO. 1 - Mix Data

Mix Designation	Cement (lbs)	Fine Aggregate (lbs)	Coarse Aggregate (lbs)	Water (gals)	W/C Ratio (by vol.)	Cement Factor (sks/yd)
A	705	200	1100	42	1.57	6.75
B	1150	550[1]	2000	87	2.0	5.65
M	380	990	505	14	0.97	5.85
L						
S						

Note: Admixture all mixes - 1 to 2 quarts "Plastiment".

1 Including 230 lbs. of 5/8" - 1/4" Lite Rock.

TABLE NO. 2

Physical Characteristics of Test Mixes

Mix	Date Cast	Specific Weight	Ultimate Compressive Strength	Secant Modulus of Elasticity
A	2/22/57	99 $\frac{lbs}{ft^3}$	5210 psi @ 56 days	2.3×10^6 psi @ 3 months 2.1×10^6 psi @ 1 year
B	3/1/57	99 $\frac{lbs}{ft^3}$	5520 psi @ 50 days	1.7×10^6 psi @ 2 months 1.8×10^6 psi @ 1 year
M	4/26/57	------	4900 psi[1] @ 28 days	1.75×10^6 psi @ 10 months
L	12/20/57	110 $\frac{lbs}{ft^3}$	6260 psi @ 28 days	1.8×10^6 psi @ $1\frac{1}{2}$ months
S	12/23/57	148 $\frac{lbs}{ft^3}$	6930 psi @ 28 days	2.6×10^6 psi @ $1\frac{1}{2}$ months

1 Ultimate strength of mix "M" was determined by crushing 2" x 2" prisms, rather than the usual 6" x 12" cylinder. The figure listed is the mean of three values ranging from 2850 psi to 5230 psi.

Specimens - Preparation for Testing

Upon their receipt at the Civil Engineering Laboratories, the specimens were prepared for test. The "A" and "B" specimens were prepared as follows:

Holes were drilled in the centers of the imbedded thumbtacks with a No. 56 drill. The holes were drilled at centers of 10.00 $\pm$ 0.01" to accept the gage points of a Whittemore strain indicator.

The specimens were capped on each end with a high strength sulphur compound, "Co-Cal" (Fig. 6). The thickness of any cap was less than 1/8". By virtue of the caps' thinness and high strength, it is thought that creep in the caps themselves may be considered of no significance.

One series A-7 Baldwin SR-4 electric strain gage was affixed to each of two opposite sides of each specimen. The strain gage was then waterproofed with "Petrolastic", a bitumastic compound.

The "A" and "B" specimens were now ready for test.

The "M" specimens were used without Whittemore points, caps, or surface strain gages. The "L" and "S" specimens were capped, but did not have Whittemore gage points, nor were electric strain gages affixed, with one exception. Four series A-1, Baldwin SR-4 electric strain gages were affixed to one "L" specimen, so that they girdled the prism at its middle. The four gages were used to measure Poisson's effect on the specimen.

Decision on Method of Loading - Constant Load vs. Load Fall-off.

The earlier investigators of concrete creep were primarily interested in its effect on concrete columns and beams under the essentially constant loads of service. Their experiments were performed on plain and reinforced concrete specimens subjected to a constant load. Obtained were relations of creep strain to time under constant load. These data are of little use in determining prestress losses, as a prestressed component is subjected to a diminishing load as the concrete yields in creep.

In order for loading conditions to be analogous to those in a prestressed beam, the concrete stress must be allowed to fall off as a direct result of concrete yielding. The first to realize this were the late Howard R. Staley and Dean Peabody, then both instructing at the Massachusetts Institute of Technology. During 1945 they conducted creep tests on plain stone concrete and gunite prisms. Each 4" x 4" x 24" prism was mounted between two steel end plates. The steel plates were tied together with four or six round mild steel bars. The bars were threaded at each end and nuts and washers affixed. Tightening of the nuts compressed the concrete specimen through the end plates and strained the round bars in tension. As the concrete yielded, the steel bars were allowed to shorten, and the load of the end plates bearing on the concrete decreased. Strain fall-off in the steel was recorded as a function of time. The steel was considered perfectly elastic at the stress levels used, and the strain fall-off was easily converted to stress fall-off. The total load in the steel bars always had to equal the total load in the concrete. Realizing this, it was a simple matter to convert stress fall-off in the steel bars to stress fall-off in the concrete prism (29, p. 229-243).

Although the method of loading described above is definitely analogous to the loading of a concrete element by concentric prestressing tendons, the analogy is distorted. The steel used for prestressing tendons is of very high tensile strength, with an ultimate ranging from 200,000 to 250,000 pounds per square inch. Working stresses of 160,000 to 180,000 psi. are common. The stressing bars used by Staley and Peabody had an ultimate strength of 60,000 psi. and were never stressed to more than 34,000 psi. (29, p. 241). Both the mild steel bars and the high tensile tendons have comparative values of

Young's modulus. This means that yielding of the concrete prisms in the M.I.T. loading units caused a much more rapid release of load than would yielding of the same magnitude in a concrete prism as normally prestressed. Or, from another viewpoint, given the same initial load, the average load sustained by a normally prestressed prism would be substantially higher than that sustained by an M.I.T. test specimen.

The answer, of course, is to use a loading mechanism in which the end plates are tied together with bars of high tensile steel, highly stressed, or, better yet, to instrument a prototype prestressed beam. The difficulties involved in following either course are not insurmountable, but they are considerable, and the solutions expensive.

It was thought reasonable and sufficient to follow the lines traced by Staley and Peabody. A compressive load would be applied to the concrete specimens utilizing mild steel bars in place of stressing tendons. The specimens would be loaded concentrically as this was thought to be a most general condition. Direct and valuable comparisons might then be made between Lite Rock and natural stone concrete under similar loading conditions.

Loading: Steel Bars and Yokes

Twelve loading units were built in the engineering shops of Oregon State College. Each unit (see Figs. 3 and 4) consists of two 2" x 1" x 6" steel end plates, or yokes, each with two 15/16" holes drilled at 4" centers, two 7/8" round mild steel bars, threaded at each end, and four nuts and four washers for the ends of the round bars. Affixed to each round bar, approximately at mid-point, and diametrically opposed to each other, are two series A-1, Baldwin SR-4 electric strain gages. These gages were waterproofed with "Petrolastic". The steel bars are also drilled for Whittemore Extensometer gage points. A No. 56 drill was used to place two sets of two holes (diametrically opposed) at 10.00 $\pm$ 0.01" centers.

Twelve specimens, six each of the "A" and "B" mixes, were placed in the loading units. All of the SR-4 electric strain gages on any one unit, both on the steel bars and on the concrete, were wired through a switching unit to a Baldwin Strain Analyser (Fig. 9). The smallest division on the analyser is one micro-inch per inch (.000001 in./in.). Two dummy gages are used. One is placed on unstressed concrete, the other on unstressed steel. These dummy gages serve to compensate for variation in electrical resistance due to temperature variation.

The six specimens of mix "A" were designated A-1 through A-6, and likewise the six specimens of mix "B", B-1 through B-6. Both the SR-4 gages and the Whittemore gage lengths on each specimen were numbered by position. For a schematic of the gage numbering system, see Fig. 10.

Unloaded "zero" readings were obtained for all electric strain gages and Whittemore gage lengths.

Standard tests were performed on both concrete specimens and steel rods to determine their respective moduli of elasticity. Values for the five concrete mixes may be found in Table No. 2. For the mild steel bars:

$$E(\text{steel}) = 29.6 \times 10^6 \text{ psi.}$$

On the day that the first twelve specimens were to be loaded, one 6" x 12" cylinder of each mix, "A" and "B", was loaded to failure in a 200,000 pound Olsen Balance Beam Type Testing Machine. The test cylinders "A" and "B" were then 56 and 50 days old, respectively. Ultimate strengths of all five concretes may be found in Table No. 2.

Three levels of initial load were applied to the test specimens. Two specimens of each mix were loaded to approximately 1/4 f'c, two to 1/2 f'c, and two to

3/4 f'c, where f'c equals ultimate compressive strength.

The loading scheme was as follows:

The "zero" readings of all gages and gage lengths were checked. A compressive load was applied to each specimen individually by a 60,000 pound Southwork-Emery Hydraulic Testing Machine (Fig. 11). The load was applied through the end yokes (Fig. 12). The nuts on the mild steel rods were taken up until the load dial on the testing machine was seen to fall off 100 pounds (as will be seen, this was insufficient nut tightening). The testing machine was then released, load being maintained in the concrete by the steel rods in tension.

All gages were read immediately upon release of the testing machine. The difference between these new-gage readings and the "zero" readings corresponds to the elastic strains due to the initial load. After all the units were loaded, they were transferred to a rack for subsequent reading and storage (Fig. 13). Gages were read at increasing intervals thereafter, attempting to keep the strain increments approximately equal in size.

After 4000 hours, the load in each unit was released and all the zero readings were checked.

Three specimens each of the "L" and "S" mixes were loaded in a similar manner in the yoke and bar units. All six specimens were loaded initially to approximately 0.4 f'c. These specimens are still under load with over 1500 hours recorded to date.

Tables No. 3-A and No. 3-B describe the loading of each of the 18 specimens loaded under the yoke and bar units ("L" and "S" specimens loaded in this manner were designated L-1, L-2, L-3, and S-1, S-2, S-4).

The loading units were subject to an elastic stress loss immediately upon release of the hydraulic testing machine. Straining the steel rods into tension, the concrete prisms lost some portion of the compressive strain originally imposed upon them by the testing machine. This is <u>not</u> analogous to the elastic stress loss that occurs in a prestressed beam when the ends of pre-tensioned tendons are released. In the latter case, there occurs no initial overstressing of the concrete. However, the writer does not believe that this short time overstressing of the test specimens caused work hardening or in any way affected subsequent creep.

Before placing any unit under the testing machine, it is necessary to know how much overload would be required to achieve a given initial stress level in the units.

TABLE NO, 3A - Loading Schedule

Specimen (* reload)	Age @ Loading	Initial Load	Initial Load % Ultimate	Duration of Load	Final Load	% Load Retained
A-1*	72 days	2630 psi	50.5	3677 hrs	1390 psi	52.8
A-2*	72	4430	85.0	3678	2690	60.6
A-3*	72	4800	92.2	3678	2780	57.8
A-4	56	1800	34.4	4099	370	20.4
A-5	56	3100	59.5	4100	870	28.1
A-6	56	2600	50.0	4102	550	21.2
B-1*	65 days	2450 psi	44.3	3699 hrs	1420 psi	58.1
B-2*	65	2310	41.7	3695	990	42.9
B-3	50	1440	26.1	4096	260	17.9
B-4*	65	3510	63.5	3695	1920	54.7
B-5	50	1970	35.6	4095	640	32.5
B-6*	65	4650	84.0	3694	2600	55.9

TABLE. NO. 3B - Loading Schedule

Specimen	Age @ Loading	Initial Load	Initial Load % Ultimate	Duration of Load	Final Load	% Load Retained
L-1	42 days	2650 psi	42.3	1482 hrs	1400 psi	52.8
L-2	42	2910	46.5	1482	1560	53.6
L-3	42	2730	43.6	1482	1365	50.0
S-1	39 days	2620 psi	37.8	1462 hrs	1700 psi	65.0
S-2	39	2310	33.3	1481	1540	66.7
S-4	39	1900	27.4	1482	1135	59.7
A-7[1]	90 days	3470 psi	66.5	1722 hrs	2300 psi	66.3
B-7[1]	66	3510	63.7	313	2545	76.8
M-1[1]	102	3320	67.8			

1 Loaded under the Olsen 30,000# testing machine.

For a hypothetical specimen that is to retain an initial load of Fo and where:

e_c = strain retained by concrete prism

E_c = Young's Modulus for concrete

A_c = cross-sectional area of prism

e_s = strain retained by steel bars

E_s = Young's Modulus for steel

A_s = total steel cross-sectional area

e_1 = strain lost by concrete on removal of testing machine

F_T = load applied by testing machine

a) For static equilibrium, the total compressive load retained in the concrete must equal the total tensile load retained in the steel.

$$F_o = e_c E_c A_c \quad \text{------------------------------}(1)$$

$$F_o = e_s E_s A_s \quad \text{------------------------------}(2)$$

b) Strain lost by the concrete on removal of the testing machine must equal strain imposed on the steel (assuming negligible yoke deflection).

$$e_1 = e_s \quad \text{----------------------------------}(3)$$

c) The load applied by the testing machine may be expressed in terms of the concrete constants and strains.

$$F_T = E_cA_c(e_c + e_1) \text{ ---------------------- } (4)$$

Substituting (3) in (4):

$$F_T = E_cA_c(e_c + e_s) \text{ ---------------------- } (5)$$

Solving (1) and (2) for e_c and e_s ,respectively:

$$e_c = F_o/E_cA_c \text{ ------------------------- } (6)$$

$$e_s = F_o/E_sA_s \text{ ------------------------- } (7)$$

Substituting (6) and (7) in (5):

$$F_T = E_cA_c(F_o/E_cA_c + F_o/E_sA_s)$$

or: $$F_T = (1 + E_cA_c/E_sA_s)\, F_o \text{ ----------------- } (8)$$

Using the values:

$E_c = 1.7 \times 10^6$ psi.

$A_c = 4.0$ in.^{2}

$E_s = 29.6 \times 10^6$ psi.

$A_s = 1.20$ in.^{2}

and substituting them in (8):

$$F_T = 1.2\, F_o$$

This means that in order to retain a given initial load in any test unit a 20% overload must be applied with the hydraulic testing machine. A more refined analysis, allowing for yoke deflection, where the yoke is considered to be simply supported and loaded by a short uniform load, yields:

$$F_T = 1.25 F_o$$

In actual operation, the "A" and "B" specimens were given an overload of approximately 30% to allow for thread slippage on the steel bars. Stress calculations during the days following initial load, showed that each unit had retained only a small fraction of its intended load. Investigation led to the finding that the washers used were not flat, but domes and that upon loading had deformed. This deformation had been sufficient to release 30% to 60% of the applied load.

The units that had been supposed at high stress, still retained enough stress to use as low stress specimens. Seven of the units were reloaded (see Table No. 3-A). The original loading scheme was followed except that the nuts were tightened until the load dial fell off five or six hundred pounds. Calculation of initial sustained stress from the first gage readings showed this nut tightening to be satisfactory.

Even before the occasion of reloading, it was decided to dispense with readings of the Whittemore Extensometer. The Extensometer was ostensibly a precise instrument, giving strains to one part in one hundred thousand (smallest division). It was found, however, that it was difficult to repeat gage readings to within one part in ten thousand. This resultant degree of precision was deemed too low and of little value.

Throughout the test program, temperature in the laboratory was maintained between 82° F. and 86° F. No attempt was made to control humidity.

Loading: Mechanical Testing Machine

It was desired to obtain information illustrating the behavior of the test concrete during the short time immediately following application of initial load. Towards this end, one specimen of each of the mixes "A", "B" and "M" has been loaded under a mechanical testing machine. Load fall-off readings were taken in time increments at first as small as 10-15 seconds.

The machine used was a 30,000 pound Olsen Universal Testing Machine of the balance beam type (Fig. 7). This machine exerts a compressive force by pulling a rigid headpiece down upon the specimen. The headpiece is lowered by four, one-inch round steel bars. The bars are strained in tension as the specimen is compressed (Fig. 8).

The action of the four steel bars of the testing machine is identical to the action of the two steel bars of the loading units described in the previous section, that is, they act as stressing tendons for the concrete.

Instrumentation of the testing machine is no problem. The testing machine gives specimen load directly, thanks to the balance beam operation.

The three specimens used in this portion of the tests were designated A-7, B-7, and M-1. Each of the specimens was placed under an initial load of approximately 14,000 pounds (3500 psi.). Specimen B-7 remained under load for approximately 400 hours. A-7 was unloaded after more than 1600 hours. M-1 remains under load at this writing and has sustained load for over 7,000 hours. Exact details of loading for these specimens may be found in Table No. 3-B.

Temperature was maintained in the mid-eighties for the duration of the tests. No attempt was made to control humidity.

CHAPTER IV

REDUCTION OF DATA

Reduction of Strain Gage Data

Data as obtained yield change in strain of stressing bars as a function of time. Utilizing the known value of Young's Modulus for the bars, this is easily converted to bar stress as a function of time. Knowledge of the physical dimensions of a unit and its statical determinancy is sufficient to reduce the above to a relationship between concrete stress and time. This is further reduced to time fall-off of stress as a percentage of the initial stress.

Moreover, since the steel bars are at no time stressed beyond their proportional limit and because Young's Modulus of steel and the physical dimensions of the units at all times remain constant, percentage stress fall-off in the concrete is always equal to percentage strain fall-off in the steel bars,

or, if we let:

F_o = initial load in unit

e_s = initial strain in steel

E_s = Young's Modulus for steel

A_s = total steel cross-sectional area

f_c = initial stress in concrete

A_c = concrete cross-sectional area

F = instantaneous load in unit at time "t"

e_t = instantaneous steel strain at time "t"

f_t = instantaneous concrete stress at time "t"

then:

$$F_o = e_s E_s A_s \quad \text{------------------------------(1)}$$

$$f_c = F_o/A_c \quad \text{------------------------------(2)}$$

substituting (1) in (2):

$$f_c = e_s E_s A_s/A_c \quad \text{---------------------------(3)}$$

also:

$$F = e_t E_s A_s \quad \text{---------------------------(4)}$$

$$f_t = F/A_c \quad \text{---------------------------(5)}$$

substituting (4) in (5):

$$f_t = e_t E_s A_s/A_c \quad \text{---------------------------(6)}$$

dividing (3) by (5):

$$\frac{f_c}{f_t} = \frac{e_s E_s A_s A_c}{e_t E_s A_s A_c} \quad \text{---------------------------(7)}$$

simplifying (7):

$$\frac{f_c}{f_t} = \frac{e_s}{e_t} \quad \text{--------------------------------(8)}$$

Therefore, percentage initial load retained in the concrete at any time "t", is equal to percentage initial strain retained in the steel bars at time "t". All curves in this group of tests, then, were plotted directly from SR-4 steel strain data.

Figures 14, 15 and 16 are plots of information as follows:

Figure 14 - a plot of Load Relaxation
vs.
Time for specimens A-1 through A-6.

Figure 15 - a plot of Load Relaxation
vs.
Time for specimens B-1 through B-6.

Figure 16 - a plot of Load Relaxation
vs.
Time for specimens L-1, L-2, L-3, S-1, S-2, and S-4.

The curves may easily be converted to give concrete deformation as a function of time. If the steel yoke deflection is neglected, change in concrete strain at any time after initial load must equal steel bar strain. Rough calculations indicate that neglect of yoke deflection will result in concrete deformations 5% to 15% too large. The larger error will occur at high stress and short time.

Reduction of Testing Machine Data

The data in these tests were obtained directly from the load dial and balance beam of the 30,000 pound Olsen testing machine. The information is plotted in

Figures 17 and 18 as follows:

Figure 17 - specimens A-7 and B-7; Load Relaxation
vs.
Time

Figure 18 - specimen M-1; Load Relaxation
vs.
Time

Again, these plots may be altered to show concrete deformation vs. time relationships, by assuming a reasonable "E" value for the stressing bars of the machine, and assuming that the head plate is infinitely rigid. No calculations have been attempted to allow for the error involved in this last assumption.

CHAPTER V

INTERPRETATION AND DISCUSSION OF RESULTS

Evidence of Instrument Error

In Figure 14 it can be seen that relaxation curves for specimens A-4, A-5, and A-6 form a close family, with values far below those of their sister curves. The same is true of the curves of specimens B-3 and B-5 in Figure No. 15. These five curves belong to the five specimens that were not reloaded in early May 1957. After the first one hundred hours, the curves for these specimens are very similar in shape to the curves for the other seven "A" and "B" specimens. But the short-time creep of these un-reloaded specimens appears to be much greater (almost twice as great) as that of the reloaded specimens. Upon unloading, it was found that the "zero" gage readings of these five specimens disagreed with their earlier "zero" gage readings by as much as 100 to 200 micro-inches per inch. This cannot be explained by any action of the units while loading or under load. It can only be explained by operator error or by some physical change having occurred in the analyser bridge in the time between the April and May loadings.

Although the curves of these five specimens are in error, they have been included. In spite of their inaccuracy, the manner in which they lie within their own families is useful evidence for many of the conclusions which follow.

<u>Sensitivity of SR-4 Gages and Analyser System</u>

The smallest division on the dial of the Baldwin Strain Analyser is one micro-inch per inch strain. Although readings may be obtained by interpolation to one-tenth of a micro-inch per inch, this degree of precision was not justifiable for the following reasons:

(1) Gage readings sometimes changed as much as five micro-inches per inch as the analyser stabilized.

(2) Body capacitance of the operator could effect a two or three micro-inch per inch change in gage reading; though on occasion it would seem to have no effect.

(3) A variation of terminals on the switching box could cause as much as twenty micro-inches per inch variation in gage reading. (For this reason, a given gage lead was always led to the same terminal throughout the test). Manner of placement of the gage lead in its terminal had little effect on gage reading (never more than 0.5 micro-inches per inch).

Relatively large strain increments occurred in the time intervals between gage readings; so it was not deemed necessary to pursue greater precision.

Shape of Relaxation Curves

Referral to any of the sets of relaxation curves (Figs. 14 through 18) will show the curves of all mixes of both aggregates to be similar in shape. At first the specimen is subject to a very rapid stress loss. The rate of stress loss decreases with time, and there is no reason to expect that given sufficient time, stress loss will not cease entirely. This would be in accord with either the Seepage or Viscous Theories.

Effect of Variables: Mix

The writer was rather surprised at first to find that creep of test specimens, series "A" and "B", varied inversely with their respective ultimate strengths (see Figs. 14, 15, 17). This is not quite so surprising after a study of the comparative mix data (Table No. 1). The weaker mix, "A", has a water-cement ratio only 3/4 that of "B" and a cement factor almost 20% richer than that of "B". These two factors, low water-cement ratio and rich mix, are normally associated with a low creep (refer back, Chapter II, Section 1).

Mix "M" has a water-cement ratio only 2/3 as great as "A" and only 1/2 as great as "B". The cement factors of the three mixes are comparable (Table No. 1). By comparison of figures 17 and 18, it can be seen that mix "M" exhibits a superior resistance to creep, relative to the two other mixes. This superior resistance might not all be due to low water-cement ratio, but may be due in part to the large amount of natural sand aggregate included in mix "B".

Effect of Variables: Magnitude of Initial Load

Lorman and Shank concluded that rate of creep is directly proportional to sustained load, at least at stresses well below ultimate (15, p. 1084 and 26, p. 493).

Referring to the curves of figures 14 and 15, in the time interval between initial load and 200 hours, it is seen that the more highly stressed specimens are creeping at the greater rate, in any one family of curves. However, sometime between 200 and 500 hours after initial loading, the relaxation curves of the more lowly stressed specimens cross the curves of their higher stressed relatives. This would indicate that, after the first few hundred hours, rate of stress fall-off in the less loaded specimens exceeds the rate of stress fall-off in the more highly loaded specimens. This is in direct contradiction to the conclusions of Lorman and Shank.

First, it must be remembered that Lorman and Shank were concerned with creep strain, not stress release due to creep strain. Secondly, they were referring to concrete sustaining a constant load, not a load allowed to decrease. As pointed out in Chapter III, stress release in the steel bars is a linear function of their strain release. Strain release in the bars is directly dependent upon the creep strain in the concrete. Then, if the conclusions of Lorman and Shank are valid, the creep strain in a concrete prism at any time should be proportional to the stress remaining in the steel bars of its unit. The relaxation curves for all specimens of any one mix should be identical, irrespective of initial stress. This is not found to be the case.

It is the writer's opinion that the behavior of the curves indicates that creep is most largely due to expulsion of colloidally adsorbed water. A specimen of high initial load must force out more water than a specimen of low initial load if they are both to lose the same percentage of prestress. But this water becomes increasingly difficult to expel as less and less of it remains. The writer believes this to be a satisfactory explanation of the crossing of the curves in Figs. 14 and 15. The short time variation may well be due to plastic flow experienced in the concrete under very high loadings.

Bulging Effect

The A-7 strain gages affixed to the concrete prisms initially indicated a shortening of the prisms corresponding to the release of tensile strain in the steel bars. After approximately 400 hours, however, the A-7 gages on all the prisms indicated a reversal of strain in the concrete (Fig. 19 is a plot of concrete strain as reported by the A-7 gages on a typical specimen). At the same time the A-1 gages on the steel bars showed a marked drop in the rate of steel stress release (see Figs. 14 and 15). After 4000 hours under load, the A-7's continued to indicate an increasing positive strain in the concrete.

When this phenomenon was first observed, the writer proposed that this seeming paradox of shortening steel and lengthening concrete is due to a "bulging effect" in the concrete. That is, the "Poisson's Effect" in the concrete is so great that the external surface of the specimen actually lengthens, though the specimen shortens as a whole. The A-7 gages are reporting this surface lengthening.

This argument is strengthened if we consider the manner in which Lite Rock concrete fails under compression.

Concrete under compressive loading normally fails in shear--indicated by the familiar cup-cone fracture.

The slender 2" x 2" x 18" Lite Rock specimens, however, failed along two mutually perpendicular planes that lie longitudinally within the specimen (Fig. 20). The 6" x 12" test cylinders also evidence this type of fracture, although ultimate failure in the stouter specimens is apparently due to shear. The failure planes run directly through the Lite Rock aggregate. These unusual planes of fracture suggest that the aggregate particles are subjected to high transverse stress, and failure is due, at least in part, to bulging. A short column of Lucite is known to fail similarly under compression.

In hopes of obtaining some information on "Poisson's Effect" in Lite Rock, four series A-1 electric strain gages were placed transversely on specimen L-1, about its midpoint. This specimen was placed under load with the other "L" and "S" specimens as previously described. The specimen exhibited a "Poisson's Ratio" of <u>0.88</u> at midpoint at initial loading. Subsequently, the cross-sectional area at midpoint rapidly diminished, until at 1500 hours it is 0.01% <u>less</u> than the specimen cross-section just prior to loading (Fig. 21).

This decreasing area is easily explained by considering shrinkage strains, but it contradicts the hypothesized "bulging effect". The paradox of apparent positive longitudinal strain in the concrete coincident with negative strain in the stressing steel remains unexplained.

<u>Stress Loss in Lite Rock Compared to Stone Concrete</u>

A glance at Table No. 4 or Fig. 16 will show that after approximately 1500 hours of loading, the "L" Lite Rock specimens exhibit, on the average, 20% more stress loss than the specimens of natural stone concrete, mix "S". This occurs under identical loading conditions and essentially the same stress level for the specimens of each mix.

It would be informative to see why this superiority of stone concrete might be expected.

The maximum strength of a natural stone concrete is limited by strength of cement paste. Best evidence of this is the manner of failure of this type of concrete. The cement paste fails around the surfaces of the individual aggregate particles, leaving the particles for the most part unfractured. The maximum strength of a Lite Rock concrete is limited by the low strength of the aggregate. Failure of Lite Rock concrete involves failure of individual aggregate particles (see Figs. 2 and 20). The aggregate acts only as a filler material and actually detracts from the strength of the concrete in proportion to its use.

TABLE NO. 4 - Percentage Initial Load Retained @ 1480 Hours
in Specimens of Comparable Initial Load

Specimen	Age @ Loading	Initial Load	Initial Load % Ultimate	% Load Retained @ 1480 Hours
A-1[1]	72 days	2630 psi	50.5	65.5
B-1[1]	72	2420	44.3	66.0
B-2[1]	72	2310	41.7	62.5
L-1	42	2650	42.3	52.8
L-2	42	2930	46.2	53.6
L-3	42	2730	43.6	50.0
S-1	39	2620	37.8	65.0
S-2	39	2310	33.3	66.7
S-4	39	1900	27.4	59.7

1 Specimens reloaded after 360 hours.

Due to the above, a Lite Rock concrete will contain a higher percentage of cement paste than will a stone concrete of equal strength, all other factors constant. Davis and other proponents of Seepage Theory agree that creep is primarily a phenomenon of the cement paste alone, and increased percentages of cement paste will result in an increase of creep (5, p. 322). This conclusion seems borne out by the curves of Fig. 16.

Although the writer has no corroborating data, it is reasonable to assume that the difference in modulus of elasticity of mixes "L" and "S" (Table No. 2) is due primarily to a like difference in the two aggregate types, i.e. that Young's Modulus for Lite Rock is substantially lower than the modulus for natural stone. If this is true, than this too may contribute to the difference in creep resistance of the two mixes. Creep within the cement paste will cause a transfer of stress from cement to aggregate (recall discussion of creep theory in Chapter II). The transferred stress induces a new and additional elastic deformation of the aggregate particles. An aggregate of lower modulus of elasticity will deform to a larger extent than one of higher modulus. This delayed elastic deformation of the aggregate will appear to the observer as additional creep. It is proposed that a relatively low modulus of elasticity for

Lite Rock would be a contributing cause of lower creep resistance.

If the Seepage Theory of creep is accepted, the higher absorption ability of Lite Rock aggregate (25, p. 23-25) would allow easier liberation of colloidally adsorbed water. Without aggregate absorption, the liberated water must either be expelled through the surface of the concrete or mechanically in the voids of the cement paste. With an absorbant aggregate, water may also be liberated to the aggregate voids, effecting an increase in rate of creep.

Lite Rock is highly porous and of very low density when compared to stone aggregate. If aggregate particles are subject to plastic flow under working loads, it would not be unreasonable to think that Lite Rock would exhibit this effect to a greater extent than the natural aggregate. This last statement is made without supporting evidence but in hope of arousing the interest of future investigators.

If reference is again made to Fig. 16, it can be seen that the curves of the "L" and "S" specimens are still diverging at 1500 hours. Mentally extrapolating the curves, an eventual divergence on the order of 25% to 30% might be expected.

Shrinkage

At no time has an attempt been made to separate the effects of shrinkage from those of creep. The writer holds that they are inseparable in concrete under load. Certainly shrinkage of unloaded concrete may be determined and is of definite magnitude for a given concrete. However, the writer contends that loading the concrete merely increases the pressure gradient responsible for water loss. Irregardless, the effects of creep and shrinkage on prestress loss are indistinguishable, and nothing is lost by considering them together.

Preference of the Seepage Theory

Referring to any of the load relaxation curves (Figs. 14, 15, 16, 17, and 18), it is seen that there were instances where the specimens actually picked up load momentarily. These load increases generally occurred during prolonged periods of wet weather, i.e. periods of high humidity. McMillan also reports this sensitivity to humidity (19, p. 252).

Positive pressure changes cannot be due to increase in water mechanically held in the concrete at normal temperatures. Only chemical reaction due to increased humidity or actual adsorption of water in the cement gel could induce an internal positive pressure. This is one more item of evidence supporting the Seepage Theory.

In the section discussing the "A" and "B" specimen curves, it was pointed out that variation in relaxation curves with degree of loading could be readily explained from the standpoint of Seepage Theory.

Three of the four possible causes of divergence of the "L" and "S" relaxation curves require acceptance of either the Seepage Theory or the Viscous Theory (refer back to section "Stress loss in Lite Rock compared to stone concrete", this chapter).

Totaling all evidence, the writer concludes that seepage of colloidally adsorbed water is a contributing factor in the creep of Lite Rock and other concretes and that it is probably the major cause.

CHAPTER VI

CONCLUSIONS

The following summary of conclusions is offered:

1. Rate and extent of creep in the Lite Rock concretes tested varies directly with the water-cement ratio of the mix and with its richness (cement factor).

2. Creep in the Lite Rock concretes is <u>not</u> directly proportional to load. After the first few hundred hours, creep rate is lower under the higher loadings.

3. Stress loss in the Lite Rock concretes occurs in the same manner as that in natural stone concretes. Rate of stress loss is at first very high, but decreases with time. The concrete appears to be approaching an equilibrium, after which no further stress loss will occur.

4. Lite Rock concrete of mix "L" is subject to a total stress loss on the order of 25% to 30% greater than that of natural stone concrete of mix "S".

5. The greater extent of creep in mix "L" as compared to mix "S" may be due to any combination of the three factors listed below:

   a) A Lite Rock concrete must contain more cement paste than a stone concrete of equal strength (all other conditions constant).

b) Lite Rock aggregate of mix "L" most probably exhibits a substantially lower modulus of elasticity than does the stone aggregate used in mix "S".

c) Lite Rock aggregate is capable of higher absorption by volume than the conventional aggregate of mix "S".

6. Seepage of colloidally adsorbed water is the major cause contributing to the creep of the mixes tested.

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APPENDIX A

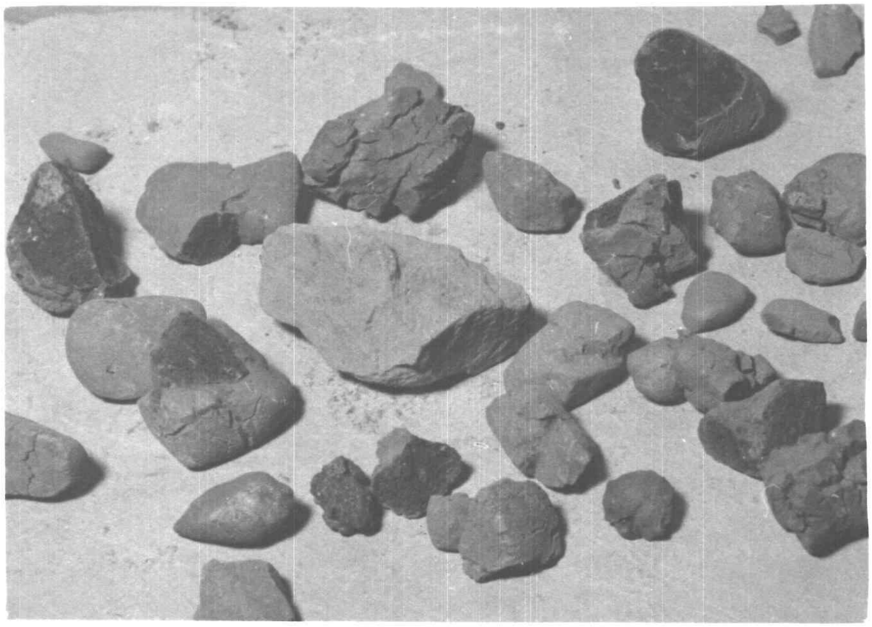


Figure 1. "Lite Rock" aggregate ($\frac{1}{2}$ actual size).

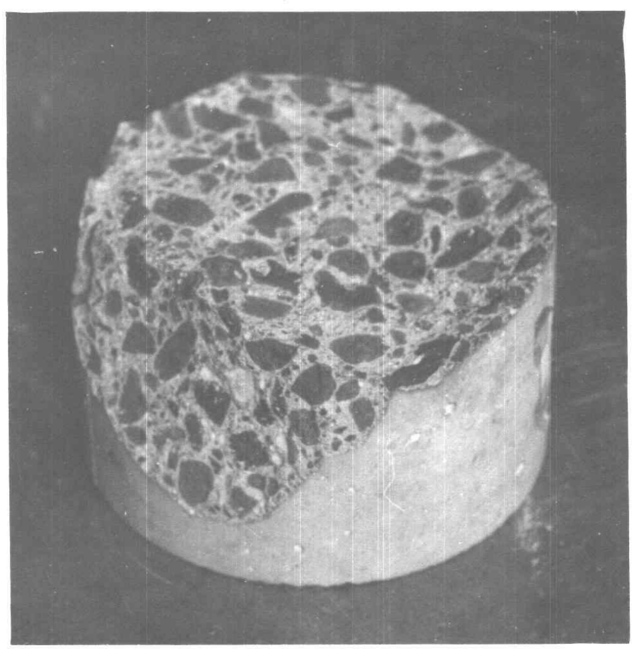


Figure 2. Cross section of "Lite Rock" concrete.

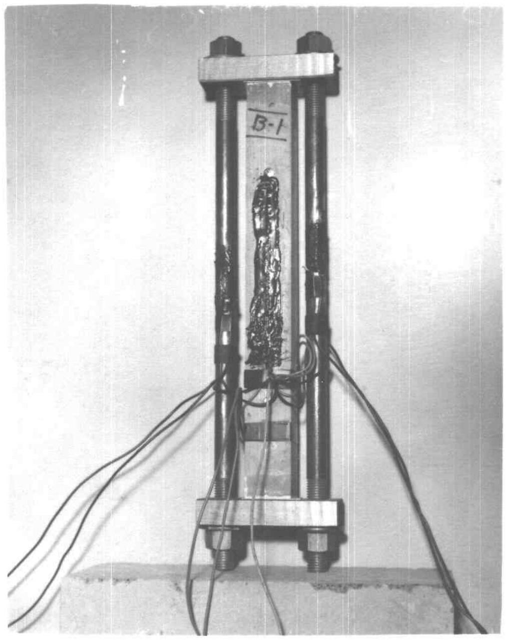


Figure 3. "B" specimen in test unit.

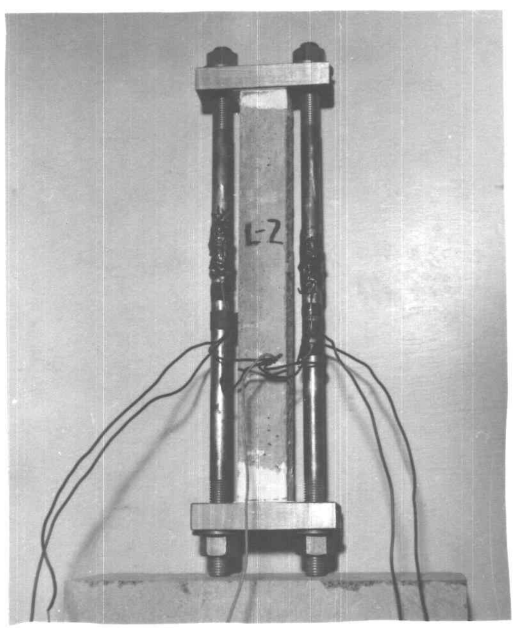


Figure 4. "L" specimen in test unit.

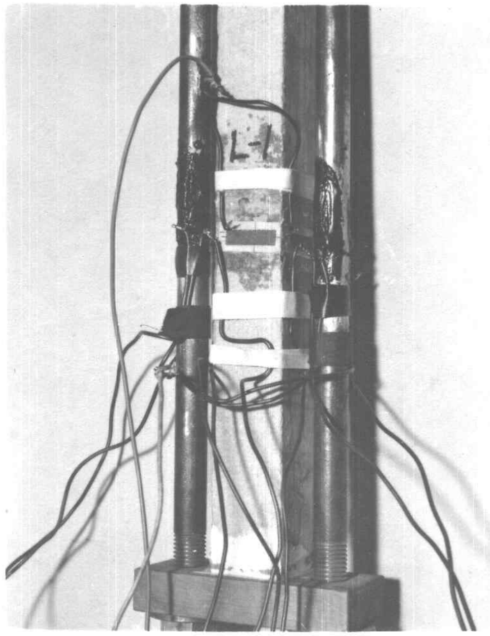


Figure 5. A-1 gages mounted to measure Poisson's Effect.

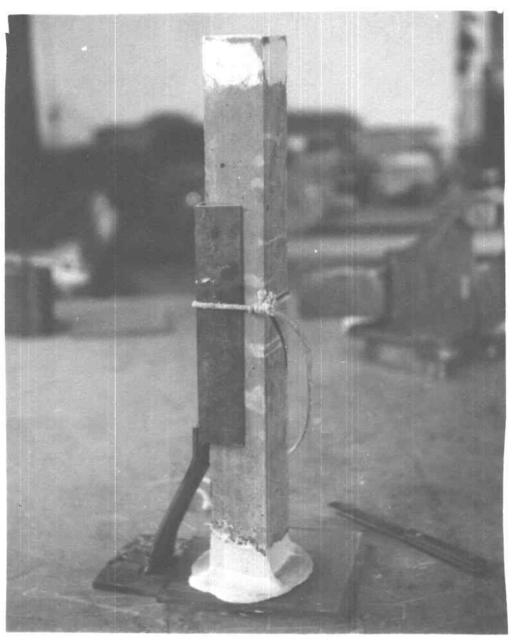


Figure 6. Capping specimen.

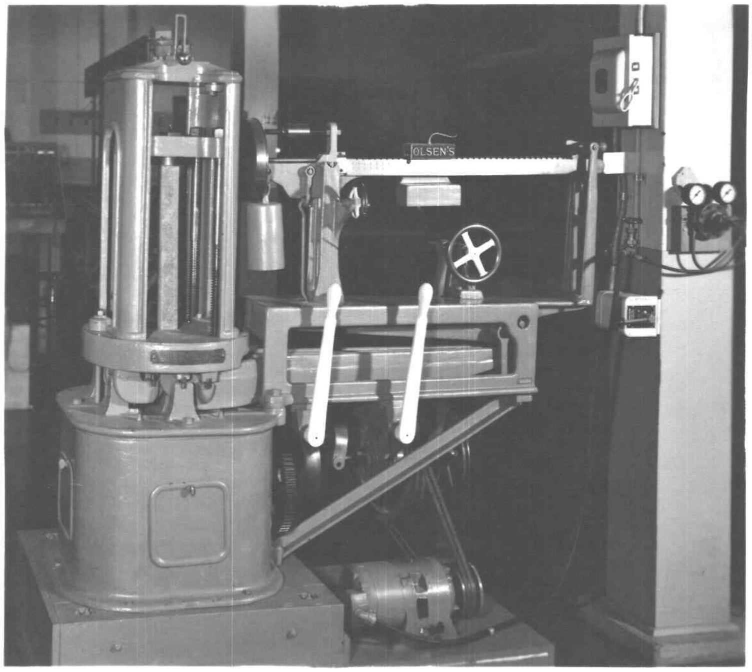


Figure 7. Specimen in 30,000 lb. Olsen testing machine.

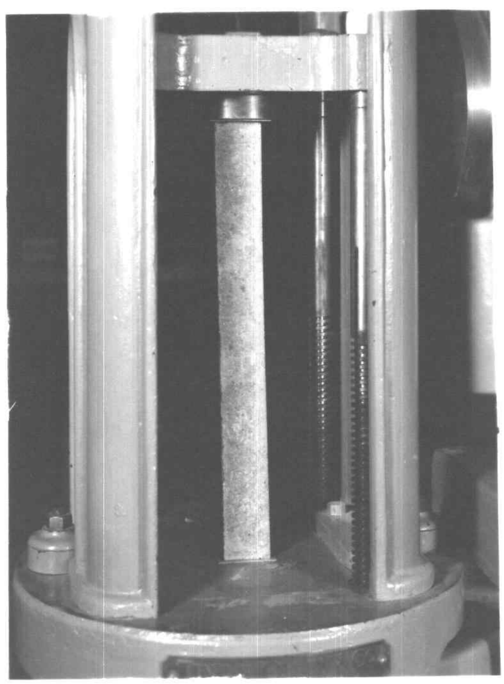


Figure 8. Loading mechanism of Olsen testing machine.

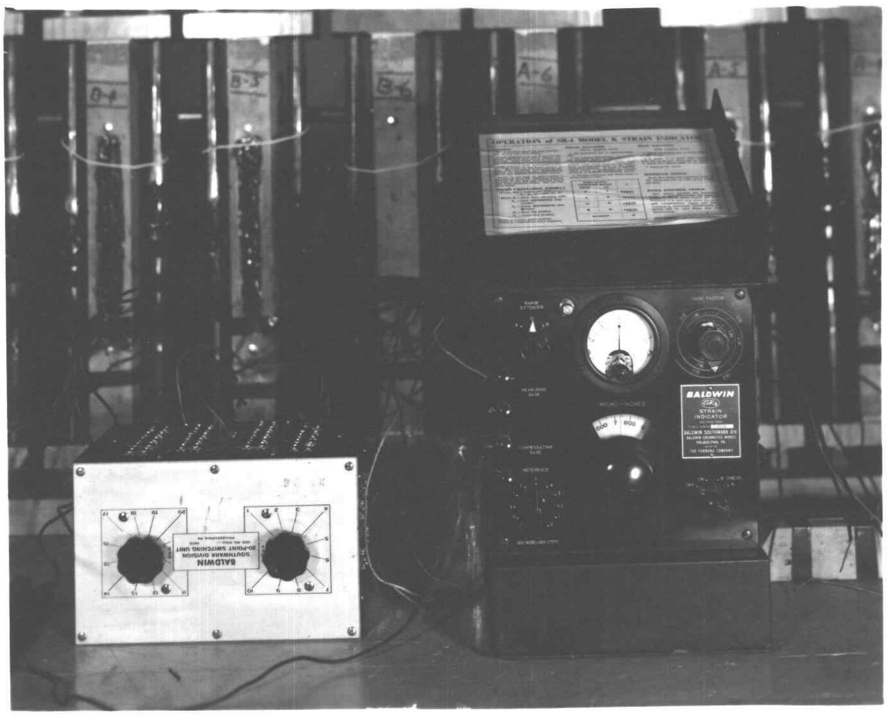
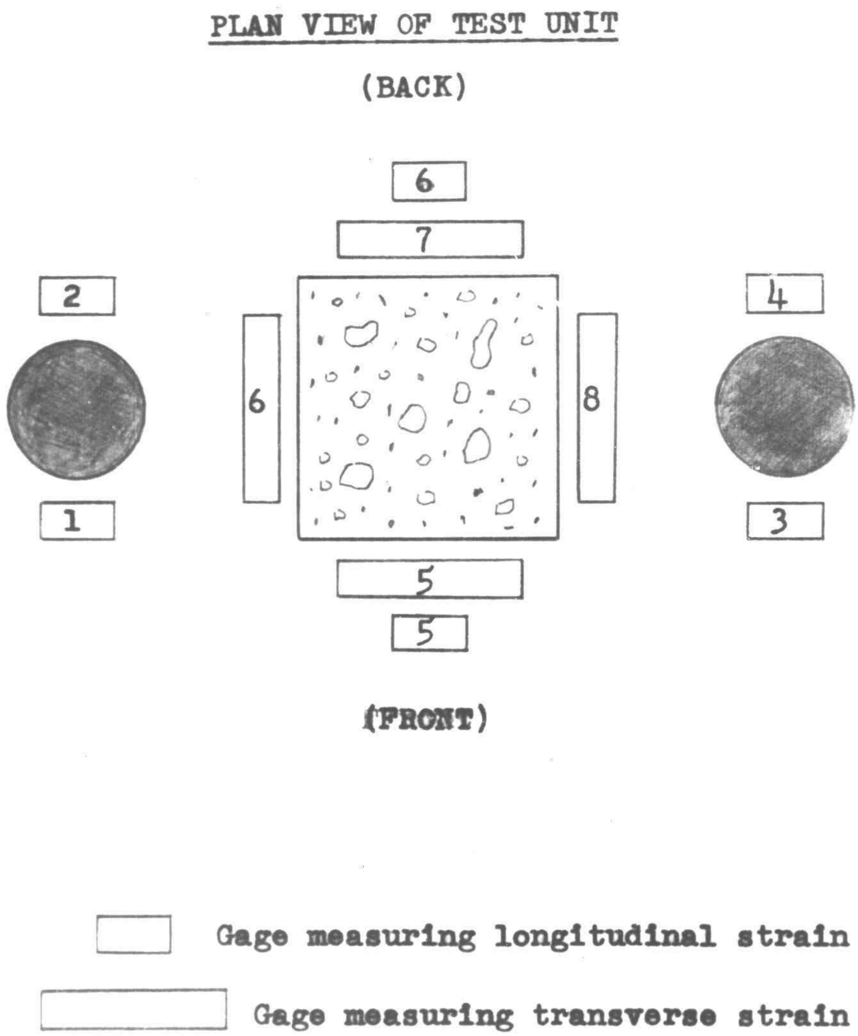


Figure 9. Switching unit and strain analyser.

Figure 10.

Schematic of Strain Gage Numbering System



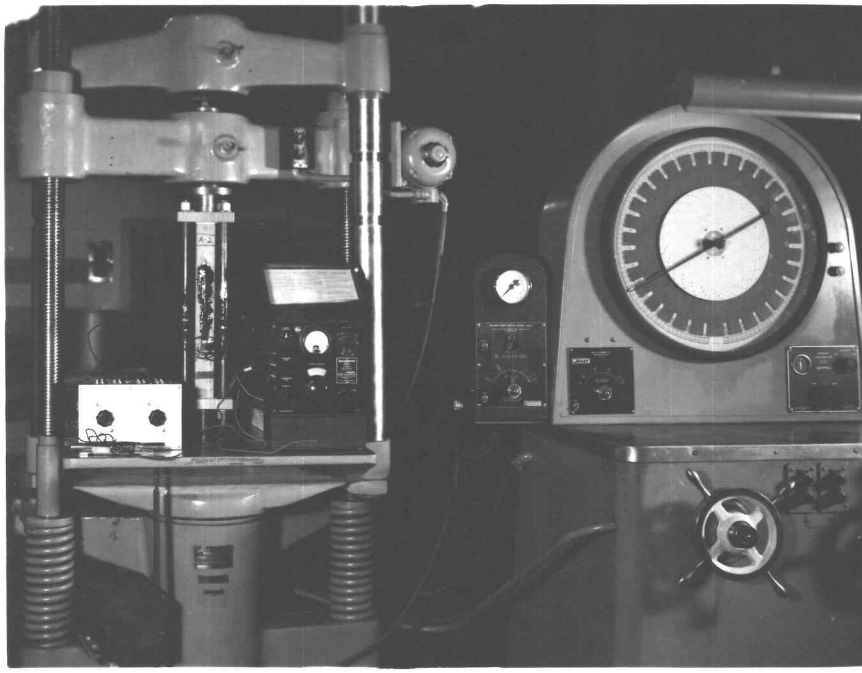


Figure 11. Southwark-Emery 60,000 lb. testing machine.

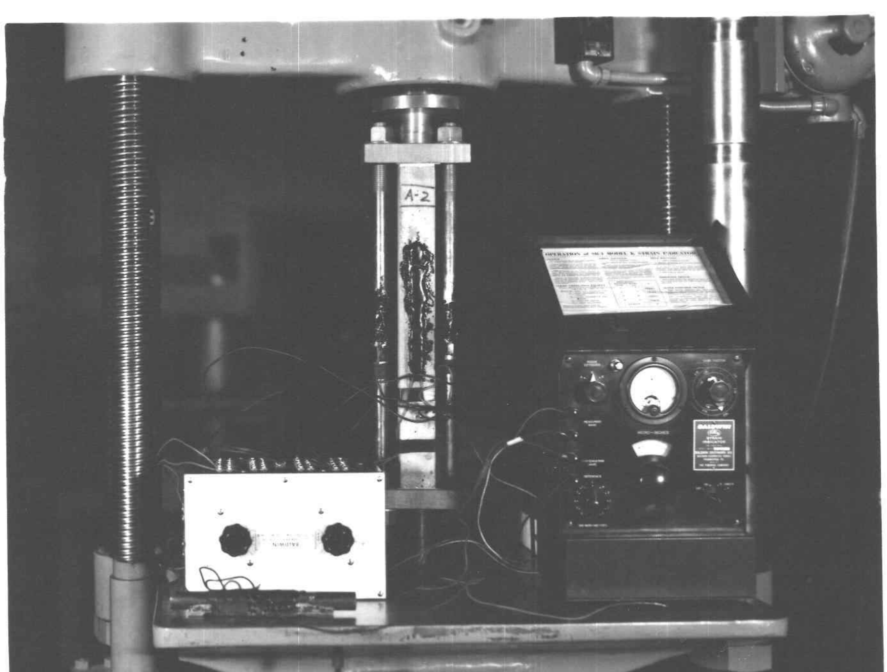


Figure 12. Initial load being applied to test unit.

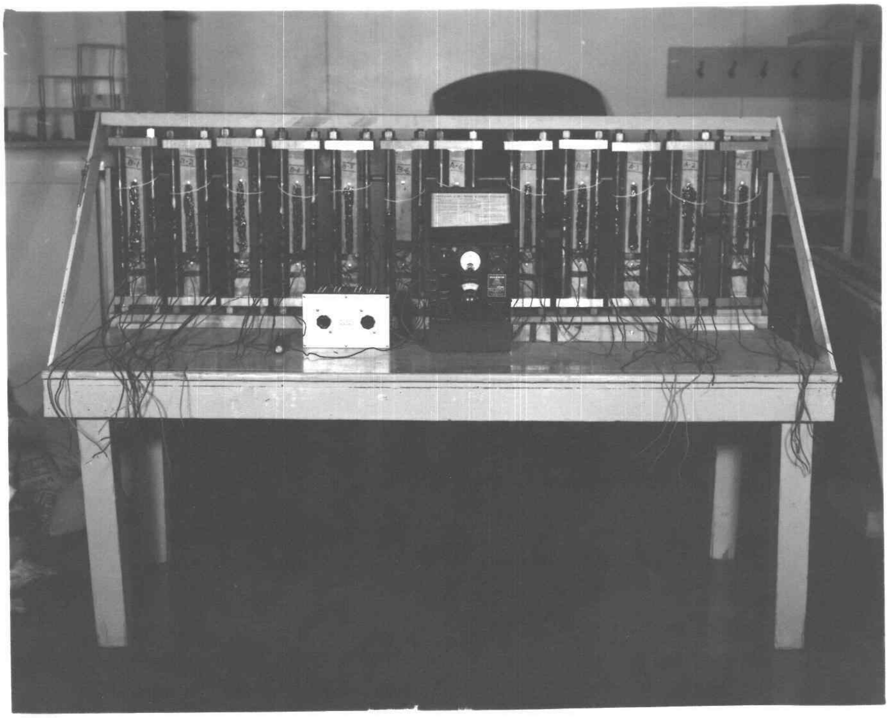


Figure 13. Test units in rack.

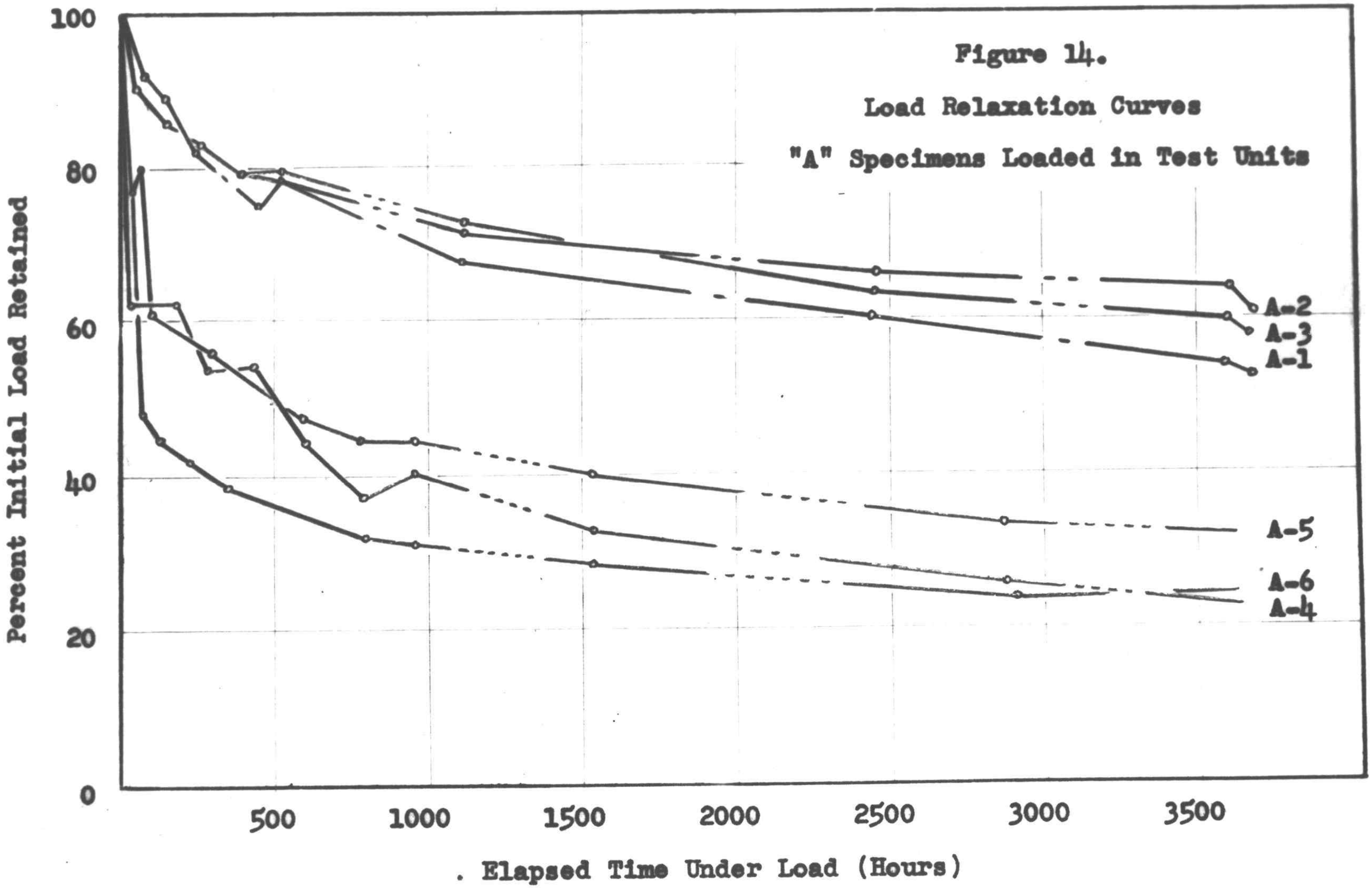


Figure 14.
Load Relaxation Curves
"A" Specimens Loaded in Test Units
Percent Initial Load Retained
100
80
60
40
20
0
500
1000
1500
2000
2500
3000
3500
Elapsed Time Under Load (Hours)
A-2
A-3
A-1
A-5
A-6
A-4

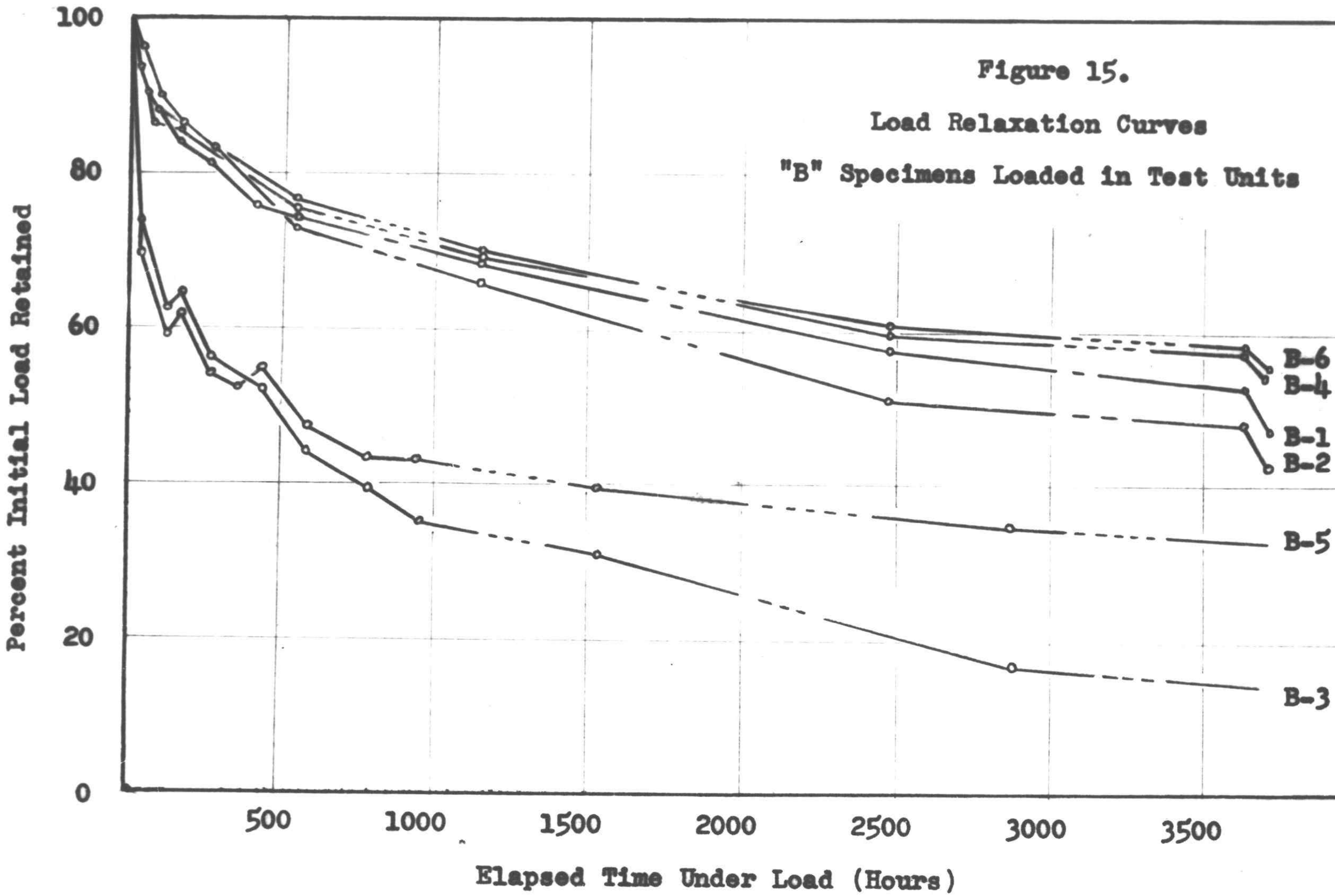


Figure 15.
Load Relaxation Curves
"B" Specimens Loaded in Test Units
Percent Initial Load Retained
100
80
60
40
20
0
500
1000
1500
2000
2500
3000
3500
Elapsed Time Under Load (Hours)
B-6
B-4
B-1
B-2
B-5
B-3

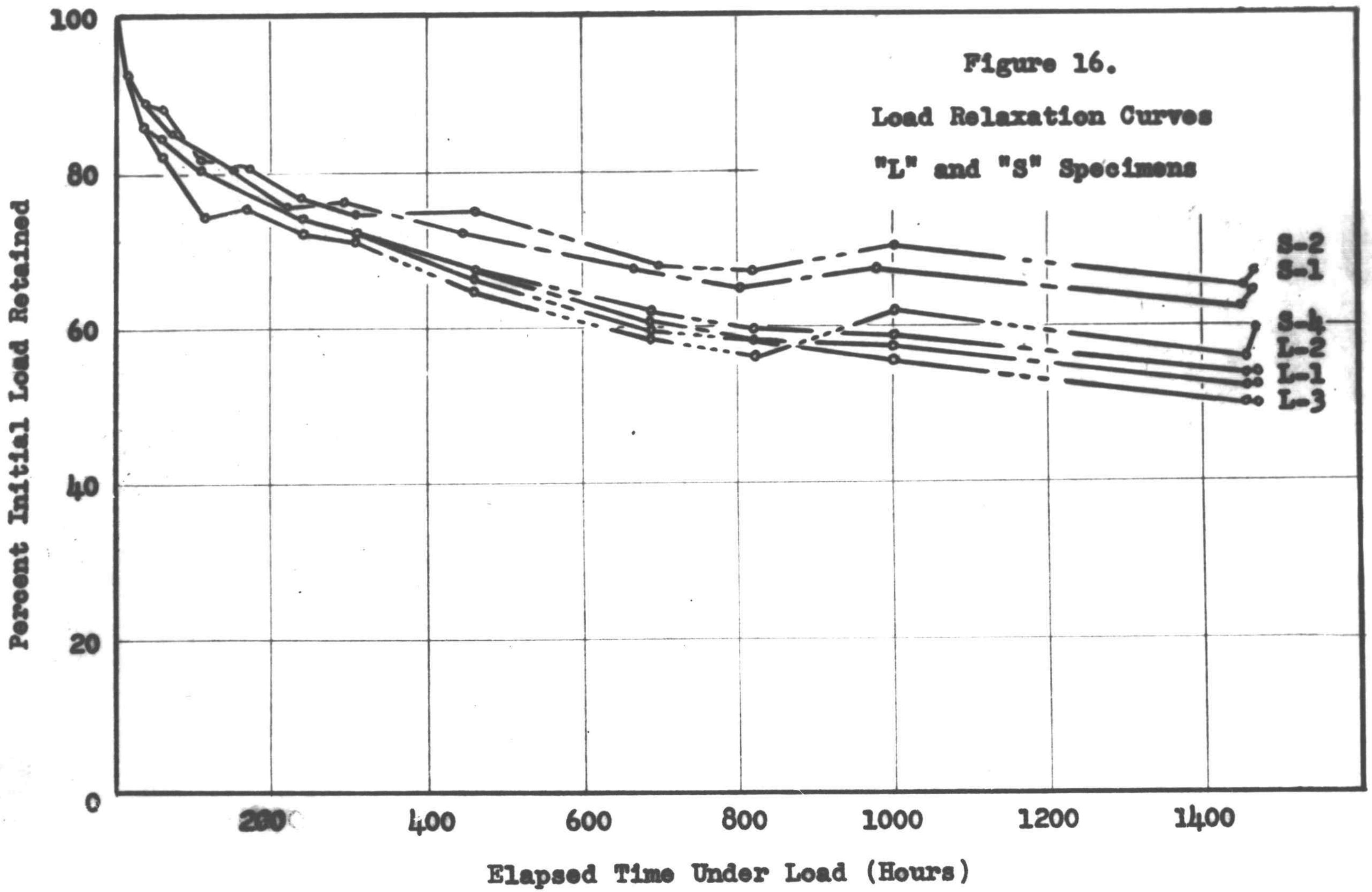


Figure 16.
Load Relaxation Curves
"L" and "S" Specimens
100
80
60
40
20
0
200
400
600
800
1000
1200
1400
Percent Initial Load Retained
Elapsed Time Under Load (Hours)
S-2
S-1
S-4
L-2
L-1
L-3

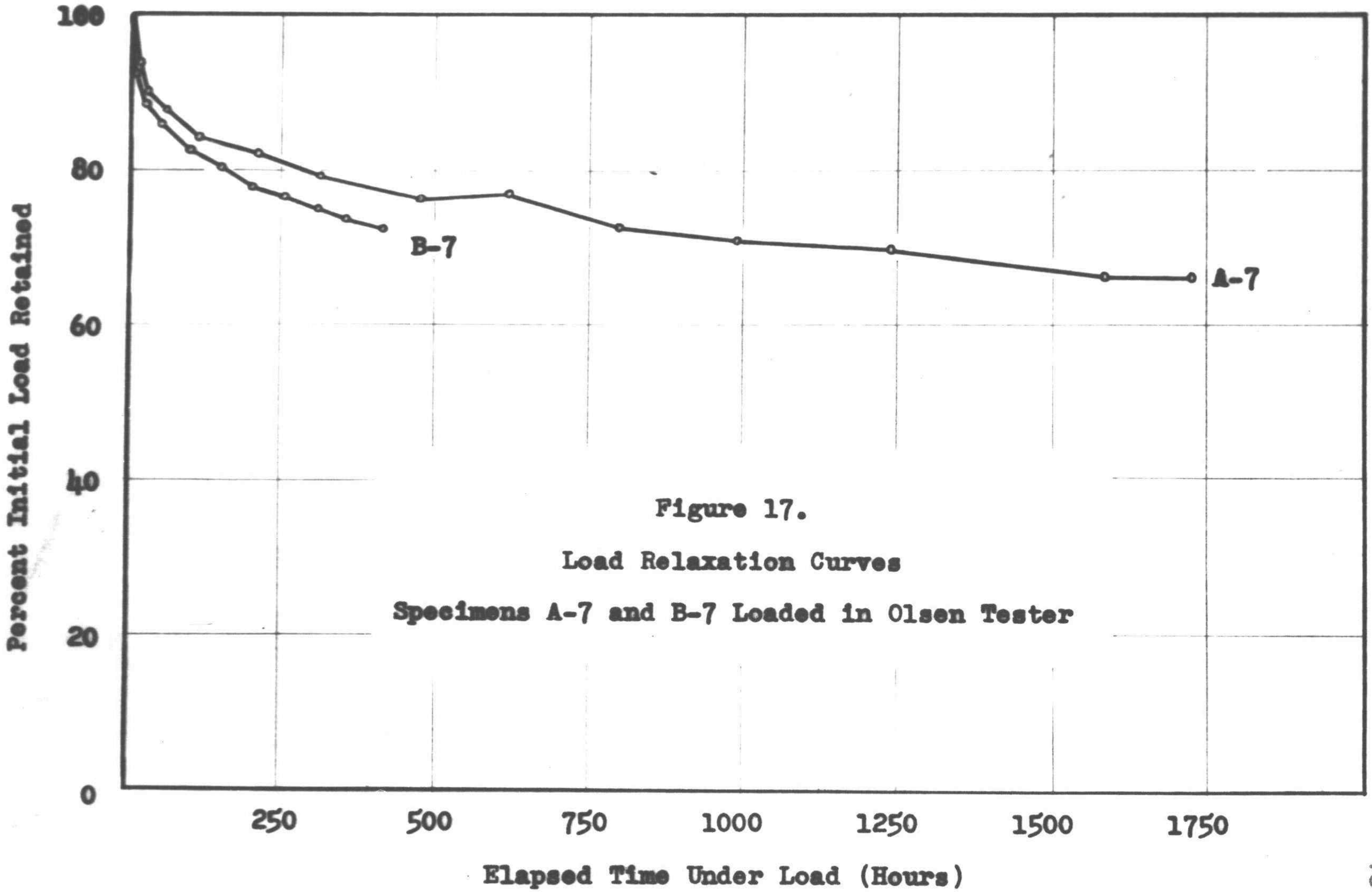


Figure 17.

Load Relaxation Curves

Specimens A-7 and B-7 Loaded in Olsen Tester

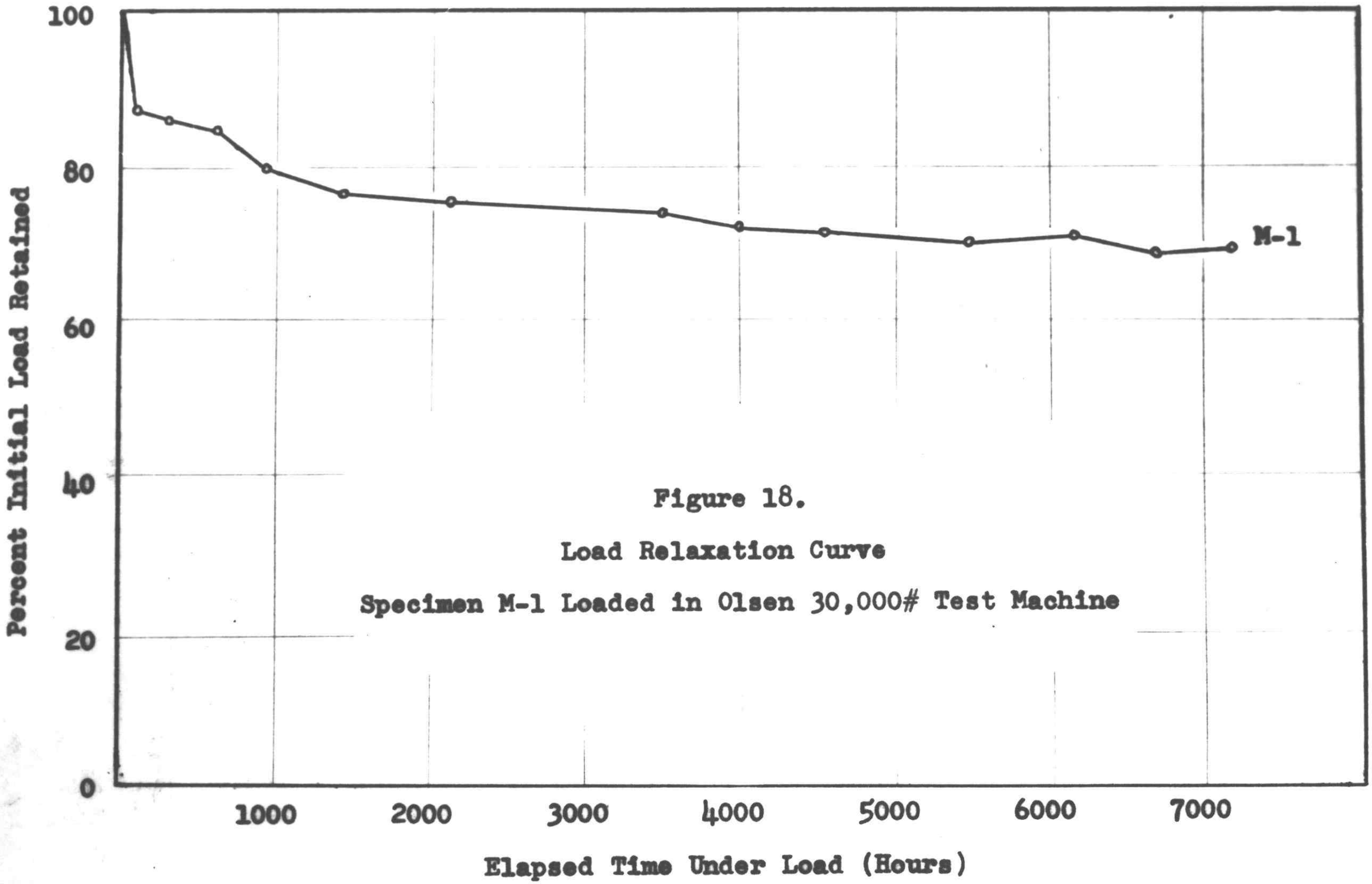


Figure 18.

Load Relaxation Curve

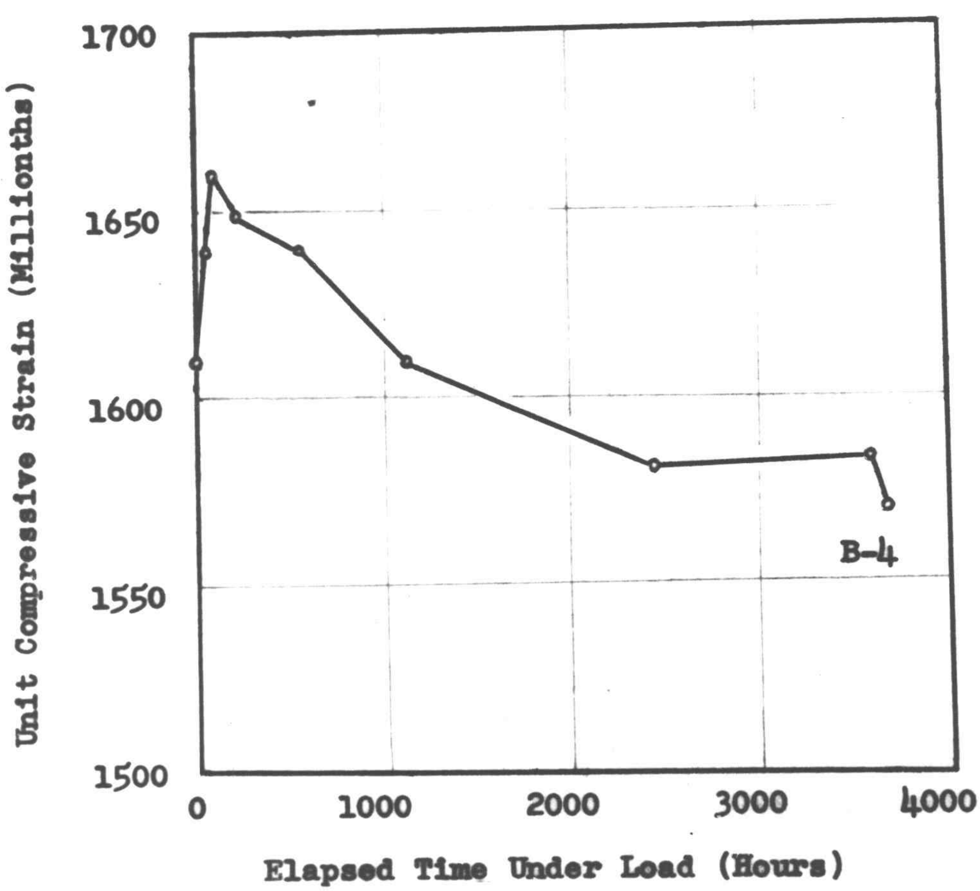
Specimen M-1 Loaded in Olsen 30,000# Test Machine

Figure 19.

Concrete Strain as Reported by

SR-4 Electric Strain Gages on

Specimen B-4 (Typical)



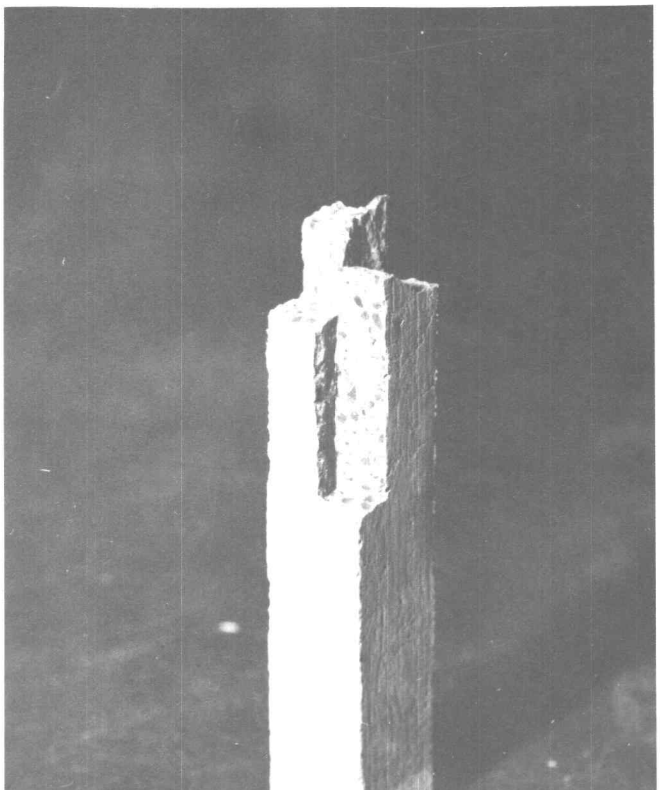
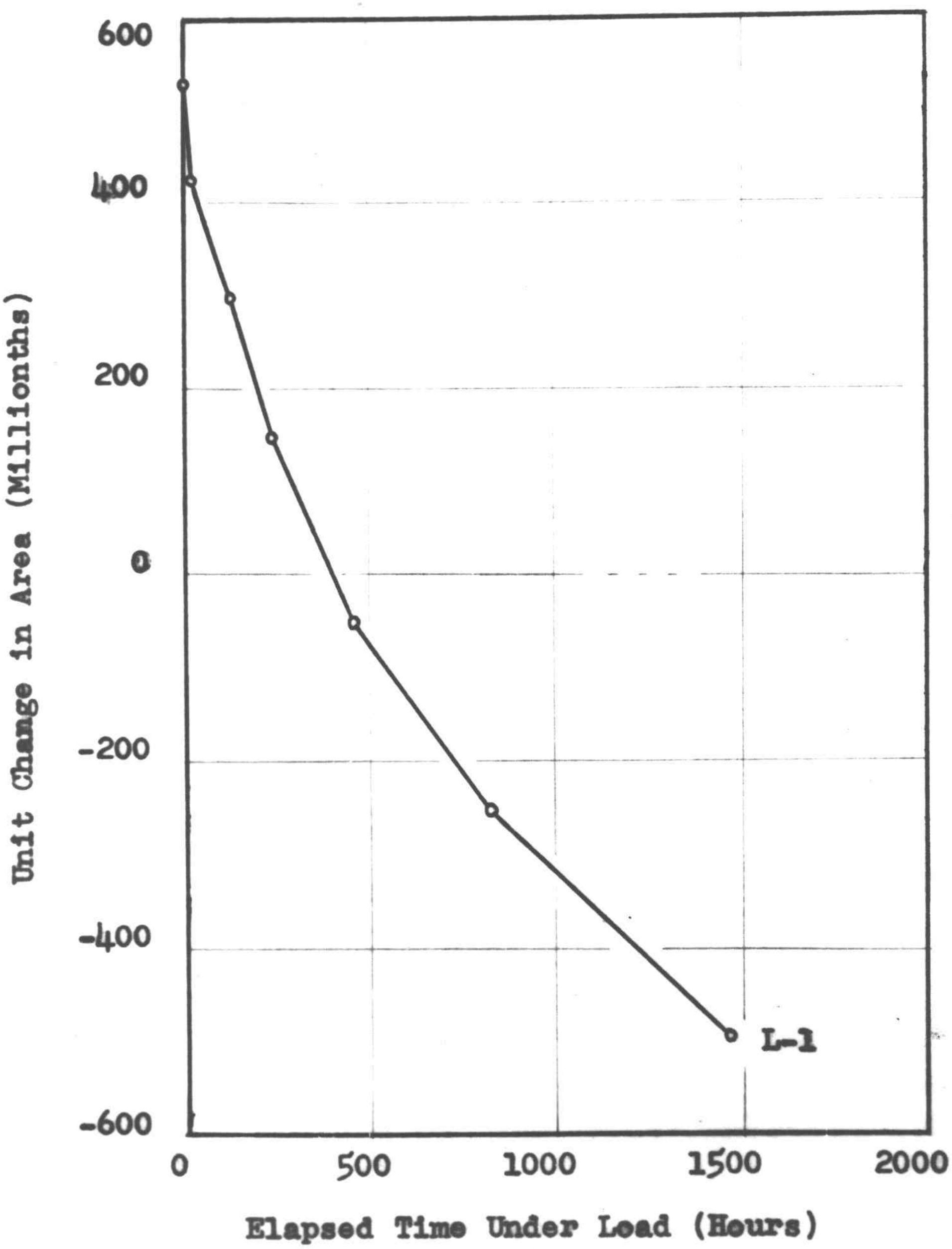


Figure 20. "Lite Rock" prism showing orthogonal fracture.

Figure 21.

Concrete X-Sectional Area at Mid-Height as Reported by SR-4 Electric Strain Gages on Specimen L-1



APPENDIX B

SAMPLE DATA SHEETS, SPECIMEN L-2

(Page One)

Gage Position (See Fig.)	Gage Reading (Micro-Inches)	Time and Date
1	03-780	"Zero" Readings
2	05-630	
3	04-620	2:00 P.M.
4	04-700	1/31/58
1	04-125	Initial Load
2	05-950	
3	04-995	2:00 P.M.
4	04-980	1/31/58
1	04-110	
2	05-940	7:30 P.M.
3	04-980	1/31/58
4	04-965	
1	04-110	
2	05-935	8:30 A.M.
3	04-975	2/1/58
4	04-960	
1	04-070	
2	05-910	1:30 A.M.
3	04-950	2/2/58
4	04-935	
1	04-070	
2	05-910	10:30 P.M.
3	04-950	2/2/58
4	04-930	
1	04-060	
2	05-895	11:00 A.M.
3	04-935	2/5/58
4	04-920	

(Page Two)

Gage Position	Gage Reading	Time and Date
1	04-040	
2	05-885	4:00 P.M.
3	04-920	2/7/58
4	04-910	
1	04-035	
2	05-880	4:00 P.M.
3	04-915	2/10/58
4	04-905	
1	04-015	
2	05-865	4:00 P.M.
3	04-900	2/13/58
4	04-895	
1	04-000	
2	05-860	10:00 P.M.
3	04-895	2/19/58
4	04-880	
1	03-970	
2	05-840	2:00 P.M.
3	04-860	2/28/58
4	04-870	
1	03-960	
2	05-840	11:00 A.M.
3	04-855	3/6/58
4	04-860	
1	03-955	
2	05-835	7:00 P.M.
3	04-855	3/13/58
4	04-855	

(Page Three)

Gage Position	Gage Reading	Time and Date
1	03-945	
2	05-815	11:00 A.M.
3	04-835	4/2/58
4	04-840	
1	03-945	
2	05-815	8:00 P.M.
3	04-835	4/2/58
4	04-840	

SAMPLE COMPILATION SHEET, SPECIMEN L-2

(1) Elapsed Time Under Load (Hours)	(2) Avg. of Four Gage Readings (Micro-Inches)	(3) Avg."Zero" Reading (Micro-Inches)	(4) Steel Strain (2) Minus (3) (Micro-Inches)	(5) % Steel Strain Retained = % Concrete Stress Retained
0	5012	4682	330	100
5.5	4998	"	316	95.7
18.5	4995	"	313	94.8
35.5	4966	"	284	86.0
56.5	4965	"	283	85.7
117	4952	"	270	81.8
170	4938	"	256	77.5
242	4933	"	251	76.0
314	4918	"	236	71.5
464	4908	"	226	68.4
684	4885	"	203	61.5
825	4879	"	197	59.6
1001	4875	"	193	58.5
1473	4859	"	177	53.6
1482	4859	"	177	53.6

COMPUTATION OF INITIAL STRESS FOR SPECIMEN L-2

Initial load in Test Unit

= Initial Steel Strain (in./in.) x Total Steel Area (sq.in.) x Young's Modulus of steel (16 s/sq.in.)

= (.000330) (1.20) (29,600,000)

= 11,700 lbs.

Initial Stress in Concrete

$$= \frac{\text{Initial load in test unit (lbs.)}}{\text{Cross-sectional area of concrete (sq.in.)}}$$

$$= \frac{11,700}{4.00}$$

$$= 2,930 \ \frac{\text{lbs.}}{\text{sq.in.}}$$