1956

Study of expanded slate aggregate for use in prestressed concrete, Progress Report No. 1, June 15, 1956

R. G. Slutter
A. Caglayan
S. Socoloske

Follow this and additional works at: http://preserve.lehigh.edu/engr-civil-environmental-fritz-lab-reports

Recommended Citation
http://preserve.lehigh.edu/engr-civil-environmental-fritz-lab-reports/1711

This Technical Report is brought to you for free and open access by the Civil and Environmental Engineering at Lehigh Preserve. It has been accepted for inclusion in Fritz Laboratory Reports by an authorized administrator of Lehigh Preserve. For more information, please contact preserve@lehigh.edu.
STUDY OF EXPANDED SLATE AGGREGATE
FOR USE IN PRESTRESSED CONCRETE

Progress Report No. 1

by

R. G. Slutter
A. Caglayan
S. Socoloske

A Report Submitted in Partial Fulfillment of the Requirements
of
C. E. 404 - Structural Research
Fritz Engineering Laboratory
Department of Civil Engineering
Lehigh University
Bethlehem, Pennsylvania

15 June 1956
Not Released for Publication

Fritz Laboratory Project No. 259
TABLE OF CONTENTS

ABSTRACT 1

INTRODUCTION 4

PART I - SUMMARY OF PREVIOUS RESEARCH ON EXPANDED SHALE AND EXPANDED SLATE CONCRETE

1. Description of Solite Aggregate 7
2. Tests on Haydite Concrete at University of Illinois 11
3. Creep Tests on Concrete at Ohio State University 13
4. Tests on Expanded Shale Concrete at Oregon State College 15
5. Tests of Prestressed Haydite Beams 17
6. Tests on Expanded Shale Concrete at University of Colorado 18
7. Information from Other Sources 20

PART II - DEVELOPMENT OF TEST PROGRAM AT LEHIGH UNIVERSITY

1. Preliminary Tests with Solite Concrete 25
2. Construction of Loading Rigs 30
3. Tests on Instrumentation System 37
4. Final Instrumentation for Test Program 48

PART III - RESULTS OF TESTS ON SOLITE CONCRETE

1. Testing Program 55
2. Revisions of Testing Program 62
3. Tests to Determine Compressive Strength 79
Table of Contents - Continued

4. Shrinkage Measurements 82
5. Creep Tests 84
6. Static Modulus of Elasticity Tests 92
7. Variation of Modulus of Elasticity Under Load 100
8. Rupture Modulus Tests 106
9. Solite Concrete and Prestressing 107

CONCLUSIONS AND RECOMMENDATIONS 118

REFERENCES 121
There has been a considerable amount of research conducted on concrete made with expanded shale and expanded slate aggregates. The research has shown that these aggregates produce one of the highest quality lightweight concretes available. A brief summary of the more important research work has been included in this report to point out the characteristics of this kind of concrete.

Most of the research programs mentioned were directed toward demonstrating that the quality of this type of lightweight concrete has led to investigations to determine if it can also be used in prestressed members. Some successful tests of post-tensioned beams of expanded shale concrete have been reported, and the results of these tests will undoubtedly encourage further investigations.

The research program now in progress at Lehigh University is studying the properties of expanded slate concrete which are of particular importance in prestressed applications. This report describes the development of a system of instrumentation for studying the performance of concrete under load, and the setting up of a test program for comparing the performance of lightweight concrete to that of conventional concrete when both are subjected to
the same constant sustained stress. The concrete under
test now consists of specimens of 3 mixes designed for a 28-
day compressive strength of 7,000 psi using type III cement
with approximately the same water-cement ratio and cement
factor. The aggregate used in the 3 mixes are as follows:

<table>
<thead>
<tr>
<th>Mix</th>
<th>Aggregate Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>sand and limestone aggregate</td>
</tr>
<tr>
<td>B</td>
<td>expanded slate aggregate without admixtures</td>
</tr>
<tr>
<td>C</td>
<td>expanded slate aggregate with entrained air</td>
</tr>
</tbody>
</table>

The concrete specimens were moist cured for 3 days and
loaded to stresses near the design stress at either 4 days or
8 days to simulate the critical conditions imposed by prestres-
sing procedures. The test has been set up so that the per-
formance of the concrete of Mix B and Mix C can be directly
compared to the performance of the concrete of Mix A under
the same conditions of loading.

The results obtained from data taken to the time the
concrete was 30 days old are reported herein. During this
period the shrinkage and creep of the expanded slate concretes
was slightly less than the shrinkage and creep of the con-
ventional concrete with limestone aggregate. The static
modulus of elasticity of the lightweight concretes was about
70% of the modulus of elasticity of the limestone concrete.
The compressive strength and the modulus of elasticity
seemed to increase with curing time at about the same rate
for all 3 concretes. Under load the rate of increase in
the modulus of elasticity was lower for the expanded slate
concrete than for the limestone concrete. The use of entrained
air in the mix for expanded slate concrete greatly improved
the workability without any apparent detrimental effects on
the properties of the concrete other than a reduction in the
rupture modulus.

The results of these tests indicate that tests of
actual prestressed concrete members are worth conducting.
The obvious advantage of the use of expanded slate or
expanded shale concrete in prestressed concrete members is a
saving of about 23% in the dead weight of the structure.
In the case of beams of lightweight concrete designed for
long spans, a saving in the amount of prestressing steel is
possible.

The final results from this testing program should make
it possible to answer the question of whether or not expanded
shale or expanded slate concretes are good enough for
prestressing.
INTRODUCTION

It is the purpose of this report to evaluate the properties of lightweight concrete made with either expanded shale or expanded slate aggregate to determine the practicality of using this lightweight aggregate in prestressed concrete structures. Part I of the report is a summary of some of the more important research work that has been conducted on concrete containing various types of expanded shale and expanded slate aggregate. Part II of the report describes the development of a means of comparing the performance of lightweight and conventional concretes under the same loading conditions. Part III consists of a progress report on tests recently started at Lehigh University on concrete made with an expanded slate aggregate known commercially as Solite.

Solite aggregate is a product of Southern Lightweight Aggregate Corporation of Richmond, Virginia. This aggregate is produced by the expansion of slate which has been heated to incipient fusion at about 2500° F. in a rotary kiln similar to the ones used in the production of cement. Previous investigation of the properties of concrete made with Solite have shown that the concrete is of high quality, and resembles in its properties the concrete made from
expanded shale aggregate, particularly the one known as Haydite. The quality of these lightweight concretes indicates that they can be used in prestressed concrete applications.

The investigations now in progress at Lehigh University are directed toward comparing some of the properties of Solite, which are of special importance in prestressing, with the same properties of conventional heavy concrete. The comparison of the following properties of Solite concrete and a conventional heavy concrete is being undertaken:

1. The rate of gain of compressive strength and the rate of increase in the modulus of elasticity with time.

2. The change of the modulus of elasticity of concrete under a constant sustained stress.

3. The amount of shrinkage.

4. The amount of creep.

5. The effect of air entrainment on the properties of Solite concrete.

This test program is being sponsored by the Southern Lightweight Aggregate Corporation of Richmond, Virginia. Lehigh Portland Cement Company of Allentown, Pennsylvania and Stressteel Corporation of Wilkes-Barre, Pennsylvania contributed materials for the tests.
The tests are being conducted under the supervision of Professor G. A. Dinsmore. The advice and assistance of Professor C. E. Ekberg, Mr. K. R. Harpel, Mr. I. J. Taylor, and Mr. C. A. Gokkent in setting up the test program is greatly appreciated. The typing of the report was done by Mrs. A. Warg.
PART I

SUMMARY OF PREVIOUS RESEARCH ON EXPANDED SHALE AND EXPANDED SLATE CONCRETE

1. Description of Solite Aggregate

The shaly slate from which Solite aggregate is produced contains roughly 55% to 60% silica, 25% to 30% alumina, and 10% to 15% carbonates and other oxides. The gases resulting from the breakdown of carbonates and impurities at high temperatures causes an expansion of the slate which produces a light, cellular aggregate which is durable and chemically inert.

The aggregate is crushed and separated into fine aggregate with a maximum particle size of 1/8 inch and coarse aggregate with a maximum size of 3/4 inch for shipment. An average sieve analysis of the aggregate received at Lehigh University was as follows:

FINE AGGREGATE

<table>
<thead>
<tr>
<th>Sieve size</th>
<th>% coarser</th>
</tr>
</thead>
<tbody>
<tr>
<td>#100</td>
<td>79</td>
</tr>
<tr>
<td>#50</td>
<td>69</td>
</tr>
<tr>
<td>#30</td>
<td>54</td>
</tr>
<tr>
<td>#16</td>
<td>40</td>
</tr>
<tr>
<td>#8</td>
<td>12</td>
</tr>
<tr>
<td>#4</td>
<td>4</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>0</td>
</tr>
</tbody>
</table>

Fineness modulus = 2.58
The unit weight of the aggregate in a dry and loose state is about 69 pounds per cubic foot for fine aggregate and about 54 pounds per cubic foot for the coarse aggregate. The unit weight of the concrete with a cement factor of 8 sacks per cubic yard and a water-cement ratio of 4 gallons of water per sack of cement averaged 118 pounds per cubic foot for concrete without entrained air and 116 pounds per cubic foot for concrete containing about 8% entrained air.

The bulk specific gravity of the aggregate as reported by the manufacturer is 1.90 to 2.00 for fine aggregate and 1.68 to 1.75 for coarse aggregate. These values were accepted and were not verified. The absorption of the aggregate is reported to be between 9% and 12% for fine aggregate and 4% to 7% for coarse aggregate. These figures were verified while working with trial mixes prior to the molding of the specimens for the test program.

Research work done on Solite concrete at the University of Maryland has shown that compressive strengths equal to
those for concrete made with conventional aggregates can be obtained for the same cement factor and water-cement ratio for any strength concrete. As a part of the tests at University of Maryland, the effect of the initial moisture content of the aggregate on the compressive strength of the concrete was studied. The tests show that the compressive strength was highest if oven dry aggregate was fed into the mixer. The 28-day compressive strength of concrete mixed with aggregate having an initial moisture content of 10% had a compressive strength about 9% lower than concrete mixed with oven dry aggregate. Similar results have been reported on investigations of other types of lightweight aggregates similar to Solite, but some investigators have shown that the reduction in strength can be reduced if mixes are carefully designed and controlled. Tests at University of Maryland also showed that the shrinkage of the concrete may be slightly greater if moist aggregates are used, but the difference is extremely small and need not be considered. The resistance of Solite concrete to freezing and thawing was found to be very good.

Since Solite aggregate is very dusty and difficult to handle without segregation in the dry state the advantages of pre-wetting the aggregate would seem to overbalance the
disadvantage of a slight reduction in strength. In the trial mixes that were made for this project, it was found that the concrete was more workable and required less mixing time if moist aggregate was fed into the mixer because of the high and variable absorption of the dry aggregate. The workability of the Solite concrete for mixes having a slump of 2 inches was satisfactory and was better than the workability of the conventional concrete designed for the same strength. The various independent investigators who have worked with expanded shale and expanded slate concrete report similar results. The rock from which Solite is produced is not greatly different from the shale which is used in the production of most expanded shale aggregates. The manufacturing process for all expanded shale and slate aggregates is essentially the same. Therefore all of these aggregates are quite similar in both chemical and physical properties except those which are not crushed after being produced. Also the performance of the aggregates in concrete is quite similar. Since a large amount of research has been done on these lightweight concretes, a review of these works will be useful in considering the results of tests on Solite concrete.
2. Tests on Haydite concrete at University of Illinois

The aggregate used in these tests had very nearly the same gradation as previously described for Solite aggregate. The coarse material with a maximum size of 3/4" had a fineness modulus of 5.89 to 6.30, and the fine aggregate had a fineness modulus in the range of 2.12 to 3.01. The specific gravity and unit weights of the material was slightly lower than the values given for Solite. The absorption of the fine aggregate averaged about 14% and was about 7% for the coarse aggregate. Sodium sulphate tests were made on the Haydite aggregate to determine the resistance to freezing and thawing. It was found that the Haydite was as durable as any of the heavy aggregates used for comparison.

The mixing procedure used was the same for Haydite concrete and sand and gravel concrete. Haydite aggregate was pre-wetted before mixing. Preliminary tests indicated that pre-wetted aggregate produced concrete of higher strength and better consistency than dry aggregate.

The workability as measured by slump and flow tests of the sand and gravel concrete could be equaled in Haydite concrete only by either using a richer mix or a higher water-cement ratio. The workability of a mix of coarse Haydite and sand was intermediate between the workabilities of the
other two mixes. The most desirable combination of fine and coarse aggregate was found to be 45% fine and 55% coarse aggregate when sand and coarse Haydite were used and 50% fine and 50% coarse when all Haydite was used. The finishing properties of the Haydite concrete were not as good as those of the stone aggregate concrete. The replacing of the fine Haydite with sand improved the finishing properties of the lightweight concrete.

The compressive strength of all Haydite concrete was as high as for conventional concrete if both had the same water-cement ratio. The rate of gain of compressive strength was the same for the lightweight and sand and gravel concretes. There were no indications that the strength of the concrete was limited by the strength of the aggregate itself within the range of mixes commonly used in structural concrete. In tests of spiral columns, the Haydite columns had lower ultimate strengths than the conventional concrete columns, but in the case of tied columns the reverse was true.

The ratio of bond strength to compressive strength at 28 days was the same for lightweight and dense concretes. This conclusion was reached on the basis of beam tests and pull-out tests of bars cast into concrete cylinders. Beam tests did not reveal any difference in the strength of
lightweight concrete and conventional concrete in shear or diagonal tension.

The modulus of elasticity of Haydite concrete ranged from 55% to 75% of the modulus for dense concrete as the mix was changed from using all Haydite to coarse Haydite and sand. The deflection of Haydite beams was about 1-1/4 times the deflection of beams of conventional concrete as a result of the lower modulus of elasticity.

The shrinkage of the lightweight concrete was about 1.6 times the shrinkage of ordinary concrete and seemed to take place over a longer period of time.

3. Creep Tests on Concrete at Ohio State University

Concretes made with many types of aggregate were loaded under sustained loads to determine the variation of the creep for like mixes using different aggregates. Natural sand and gravel was used as the standard of comparison for the concretes made with other aggregates. The test data was obtained by using temperature and humidity control cylinders instead of attempting to maintain constant temperature and humidity. This method was successful.

The data taken was reduced to a formula which would express the creep in terms of coefficients which depend on the age at loading, type of cement used, and type of aggregate.
The following formula which gives good results for a period of 1 year for sustained loads within the working range was developed:

\[ y = C \sqrt[3]{x} \]

- \( y \) = creep in millionths per psi of sustained axial stress
- \( x \) = duration of loading in days
- \( a \) and \( C \) are empirical coefficients derived from tests

The formula for concrete of natural sand and gravel loaded in air at age 28 days becomes \( y = 0.130 \sqrt[3]{x} \). The results of tests on Haydite concrete were neither extensive nor dependable. The formula determined for Haydite concrete was \( y = 0.220 \sqrt[3]{x} \). This indicates that the expected creep of Haydite concrete is about 1.4 times the creep of sand and gravel concrete at the end of 1 year. Concretes made with different kinds of dense aggregates, such as granite and basalt, creep more than concrete made with sand and gravel aggregate. None of them creep as much as Haydite. However, the creep of Haydite concrete is not much larger than the creep of concrete made from some types of stone aggregate. The variation in creep of concrete made from different aggregates seems to be due to the gradation of the aggregate, shape of the particles, and the surface texture rather than any difference in the strength of the particles. Tests made
on concrete containing glass as aggregate was found to creep excessively. Since glass compounds will form at the temperatures at which expanded shale aggregate is heated in the expansion stage of manufacture, the presence of glass particles could be a reason for greater creep of expanded shale concrete.

4. Tests on Expanded Shale Concrete at Oregon State College

Several different kinds of expanded shales and slates were tested. The tests indicated that there was little difference in the properties of the different aggregates or their performance in concrete. One of the expanded shales known commercially as Rock-Lite differs from the others in that it is not crushed after being produced so that the surface of the particles is covered by an impervious layer of fused shale. The other types of expanded shales and slates were crushed after being produced and therefore presented an exposed surface which was porous.

In proportioning the richer mixes, it was observed that the percentage of coarse aggregate had to be reduced to obtain the required strength for a given water-cement ratio because the strength of the aggregate itself became a limiting factor. The workability of the mixes was satisfactory without the use of wetting agents. However, the use of
wetting agents was recommended because they permit a reduction in the quantity of water and thus an increase in compressive strength. For richer mixes the benefit of wetting agents was especially important. It was noticed that the lightweight concretes required more vibration than similar mixes of conventional concrete. The reason for this is the difference in weight of the particles. For the same reason lower values for the slump test were obtained for lightweight concrete than for conventional concrete having the same workability.

The compressive strength of the lightweight concretes usually exceeded the strength of the sand and gravel concrete with which they were being compared. The lightweight concretes had satisfactory bond strength and flexural strength, but their strength in diagonal tension in beams was lower than that of the conventional concrete. The lower diagonal tension resistance was found in tests of both unreinforced beams and rupture modulus specimens.

The modulus of elasticity of the expanded shale concretes averaged about 50% as high as the modulus of elasticity of conventional concrete. The lightweight concrete had a lower shrinkage in 28 days than did conventional concrete. The abrasion resistance was much lower for light-
weight than for conventional concrete. The strength gain after 7 days of curing was less for lightweight than for conventional concrete.

5. Tests of Prestressed Haydite Beams

The Carter-Waters Corporation of Kansas City, Missouri and Prestressing, Inc. of San Antonio, Texas have cooperated in the testing of post-tensioned beams of Haydite concrete. Several "I" beams designed for a roof system having a depth of 20 inches and a span of 20 feet were tested under various loading conditions for a period of 120 days. The Haydite concrete mix was designed for a 28-day compressive strength of 5000 psi. The mix was 6.75 sacks of cement per cubic yard with 7-1/2 gallons of water per sack of cement. The actual compressive strength from test cylinders were as follows:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>2 days</td>
<td>3,000 psi</td>
</tr>
<tr>
<td>14 days</td>
<td>4,500 psi</td>
</tr>
<tr>
<td>28 days</td>
<td>6,000 psi</td>
</tr>
</tbody>
</table>

Both grouted and ungrouted beams were tested and found to behave elastically up to the design load. The modulus of elasticity of the concrete was $3.24 \times 10^6$ psi. at time of test as computed from beam deflections. This value was deemed high enough for prestressed concrete. The decrease of the modulus of elasticity under load was not excessive. It
decreased to $1.73 \times 10^6$ psi after 121 days of continuous loading, but the recovery of the modulus of elasticity after release of load was satisfactory.

The ultimate strength of the grouted beams was considerably higher than that of the ungrouted beams. The ultimate load for the ungrouted beams was very close to the cracking load while the ultimate load of the grouted beams is higher above the cracking load than for the ungrouted beams.

The losses in prestress at the end of 120 days was 25%. The portion of the loss due to creep and shrinkage was believed to be 17%. This amount of loss due to creep was not considered to be too high.

6. Tests of Expanded Shale Concrete at University of Colorado

These test results demonstrate that the performance of expanded shale concrete in prestressed concrete beams is quite good. The expanded shale used in these tests is known as Idealite and is produced by Great Western Aggregate Co. The outstanding characteristic of this aggregate is that the surfaces of the particles are sealed by fusion so that the aggregate has a low absorption and therefore produces a good quality of concrete which is lighter in weight than most expanded shale concretes. The weight of the expanded
shale concrete was 99.8 pounds per cubic feet and had a 28-day strength of 5000 psi.

The compressive strength of the expanded shale concrete was usually slightly higher than conventional concrete having nearly the same cement factor and water-cement ratio. The strength of the lightweight concrete in bond and shear was reported to be as good as for conventional concrete of comparable compressive strength. Modulus of rupture of the expanded shale concrete was 7.4% of the compressive strength. Modulus of rupture of conventional concrete averaged 10.9% of compressive strength. The fact that the expanded shale aggregate is weaker in tension than the conventional aggregate seems to offer satisfactory explanation for the lower modulus of rupture of the lightweight concrete. It was observed that rupture modulus specimens of lightweight concrete always fractured through the aggregate whereas the conventional aggregate sometimes was pulled out of the paste upon fracture.

It was observed that the ultimate strain of the lightweight concrete corresponded to the strain at ultimate load very nearly. The failure of the compression test specimens occurred at ultimate load, and the failure was usually a vertical splitting fracture. In conventional concrete the failure occurs at a strain which exceeds the strain at
which ultimate load is reached and the fracture produces a cone at the base of the cylinder.

The amount of camber in the prestressed beams was quite large as compared to the camber in the beams of conventional concrete as a result of the lower modulus of elasticity of the lightweight concrete. The modulus of elasticity of the expanded shale concrete ranged from $2.5 \times 10^6$ psi to $3.0 \times 10^6$ psi as compared to $3.6 \times 10^6$ psi to $4.3 \times 10^6$ psi for the conventional concrete. The modulus of elasticity of the lightweight was 70% of that for conventional concrete.

The observed shrinkage of the two concretes was about equal for both and did not exceed 0.0003 inches per inch. The amount of creep of both concretes seemed to be equal also, but the period of measurement was only 2 months which may not be a long enough period to be conclusive.

7. Information from Other Sources

The reports of most investigators indicate that pre-wetting of aggregate and the use of air entraining are well established methods of insuring good consistency and workability in expanded shale concretes. For the rich mixes used for prestressed concrete, these recommendations also seem to apply. It has been demonstrated that the amount of
entrained air used in lightweight concrete should be about double the amount which is used in conventional concrete of a similar mix because of the amount which is absorbed into the aggregate. The same percentage reduction in compressive strength for lightweight and conventional concrete will occur when the lightweight has twice as much entrained air.

Investigations have shown that the entrained air will reduce the modulus of elasticity only by the percentage by which compressive strength has been reduced for conventional concrete. Therefore the reduction in modulus of elasticity is of the order of 10\% when air entraining is used. There is no evidence to show that this same relationship will not be the same with lightweight concrete. Air entraining has little or no effect on shrinkage and creep of conventional concrete and apparently does not affect the shrinkage and creep of lightweight concrete either.

There are several sets of test results which indicate that shrinkage of lightweight concrete takes place over a longer period of time than in conventional concrete. Tests at University of Missouri show that although conventional concrete will reach 90\% equilibrium in about 30 days, it requires about 90 days for Haydite concrete to reach 90\% equilibrium. Since shrinkage and creep are related phenomena, it is to be expected that creep also will continue over a
longer period of time in expanded shale concrete.

Tests at University of Missouri of post-tensioned slabs made with Haydite concrete were recently conducted. The deflections of the Haydite concrete slabs were much larger than those of the slabs of conventional concrete. The amount of deflection was considered to be excessive for the Haydite slabs, and it was recommended that the thickness of the slabs be increased. Other tests have been reported in which deflections of Haydite slabs were excessive. Deflections of Haydite beams usually are of the order of 1-1/4 times the deflections of similar beams made of conventional concrete. This amount of deflection is usually not excessive. The importance of these test results in the case of prestressed concrete may be that thin prestressed sections will exhibit excessive camber deflection requiring that thicker sections be used.

A possible explanation for the difference in deflection behavior between lightweight beams and slabs may be found if the assumption that thin sections of concrete act more nearly like a homogeneous material than do deeper concrete beams is valid. If the slab acts as a homogeneous material then the deflection of the slab is inversely proportional to the modulus of elasticity of the concrete. However, it is known that the deflection of a concrete beam is not inversely
proportional to the modulus of elasticity of the concrete. Professor Maney proposed the following equation to show that in beams the deflection does not change in a linear manner when the modulus of elasticity of the concrete varies:

\[ D = K \frac{L^2}{d} (e_{\text{steel}} + e_{\text{concrete}}) \]

In this equation, \( D \) is the deflection of a reinforced concrete member, \( L \) is the span of the member, \( e_{\text{steel}} \) is the strain in the steel, \( e_{\text{concrete}} \) is the strain in the concrete, and \( d \) is the depth of the beam or slab. Since the strain in the steel will be nearly the same regardless of the variations in the strain of the concrete due to changes in the modulus of elasticity of the concrete, the deflection of a member will not increase linearly as the modulus of elasticity of concrete is lowered. Thin prestressed slabs with a low percentage of steel apparently behave more like a member of homogeneous material, and deflect amounts which are nearly inversely proportional to the modulus of elasticity of the concrete in the slab. More research work is needed for a complete explanation for the deflection behavior of expanded shale slabs.
Although the research done so far in using expanded shale concrete in prestressed concrete members is limited, the results of the tests completed indicate that good quality expanded shale aggregates are suitable for prestressed concrete members. There is not enough information available for estimating the loss of prestress in expanded shale beams.
PART II
DEVELOPMENT OF TEST PROGRAM

1. Preliminary Tests with Solite Concrete

The Solite aggregate was received in bags weighing about 100 pounds. The coarse and fine aggregate was bagged separately. Because the aggregate became wet during shipment, the fine aggregate was spread out in a bin to dry. The coarse aggregate was left in the bags until shortly before using. It was noted that the coarse aggregate had not segregated much in shipment.

The first trial mixes were made using dry aggregate, but with a rich mix and a low water-cement ratio, it was extremely difficult to obtain a satisfactory mix. The amount of absorption varied greatly from batch to batch, and some of the batches could not be used without adding more than the calculated amount of water. The characteristic of the poorer batches was that the cement paste formed in balls around particles of coarse aggregate. It was very difficult to finish the top surface of the cylinders. After removing forms the cylinders were found to be honeycombed even though the concrete had been vibrated in the cylinders. The slump of these poor batches was about 1/2 inch.
The fine aggregate was then wetted and mixed several times. The moisture content was kept at about 8% to 10%, and several moisture determinations were made before each set of batches. The same mix which had been previously made with dry aggregate was made using the damp fine aggregate and dry coarse aggregate. The resulting concrete was much more workable than that produced when using all aggregates dry. The consistency of the concrete was more uniform.

The proportions of fine and coarse Solite were varied to determine the best proportions for a mix using a cement factor of 8 sacks per cubic yard and a water-cement ratio of 4 gallons per sack of cement. The proportions originally recommended by Mr. Duey of Southern Lightweight Aggregate Corporation were found to be satisfactory. These proportions for one cubic yard of concrete were as follows:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>8 sacks of Type III</td>
</tr>
<tr>
<td>Fine Solite</td>
<td>1250 pounds</td>
</tr>
<tr>
<td>Coarse Solite</td>
<td>850 pounds</td>
</tr>
<tr>
<td>Water</td>
<td>268 pounds plus allowance for absorption</td>
</tr>
</tbody>
</table>

This mix resulted in a 2" slump, and the workability was quite good. The finishing properties were as good as those of sand and stone concrete.

Several mixes were made using sand and coarse Solite starting with proportions recommended by Mr. Duey. The
cement factor and water-cement ratio were the same as for the mix of all Solite. A fine sand having a fineness modulus of 2.50 was used in the trial mixes. It was difficult to obtain a mix having a suitable workability from this combination of aggregates. The best proportions were found to be the following:

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>8 sacks</td>
</tr>
<tr>
<td>Sand</td>
<td>1250 pounds</td>
</tr>
<tr>
<td>Coarse Solite</td>
<td>1100 pounds</td>
</tr>
<tr>
<td>Water</td>
<td>268 pounds plus allowance for absorption</td>
</tr>
</tbody>
</table>

This mix contains a high percentage of sand, and it was felt that the weight of this concrete would be so near to that of conventional concrete that the saving in weight would not be significant. Therefore no further work was done using the combination of sand and coarse Solite.

An air entrained mix was made using the same proportions as were used in the all Solite mix. An amount of Darex AEA was added in the mixing water which would entrain about 4-1/2% of air in conventional concrete according to the manufacturer's recommendations. This amount of air did not produce any improvement in the workability of the concrete. Since a mix of the richness being tested would be used for prestressed concrete, it was considered desirable to use a minimum of entrained air so that there would not be much reduction in the compressive strength or the modulus of
elasticity. However, the effect of the entrained air on compressive strength can usually be compensated for by a slightly lower water-cement ratio. A mix using an amount of Darex AEA which would entrain about 8-1/2% of air in conventional concrete was tried. The amount of air in the paste can not be easily determined because part of the air is absorbed in the aggregate. Equipment for determining the amount of air in the concrete was not available, and therefore no attempt was made to measure the actual amount of entrained air. The second air entrained mix had greatly improved workability and finishing properties compared to the other mixes of Solite concrete. Compression tests showed that the strength of the air entrained concrete was higher than the strength of the other mixes. Investigations have shown that any reduction in modulus of elasticity as a result of air entrainment will be in about the same proportion as the reduction in the compressive strength. If the compressive strength of the air entrained concrete is actually higher than for the plain Solite concrete, then the modulus of elasticities should have the same relationship. The improved workability and finishing properties of the air entrained mix was considered justification for testing it in place of the sand and Solite mix.
The following proportions were used for the air entrained mix:

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>8 sacks of Type III</td>
</tr>
<tr>
<td>Coarse Solite</td>
<td>875 pounds</td>
</tr>
<tr>
<td>Fine Solite</td>
<td>1150 pounds</td>
</tr>
<tr>
<td>Water</td>
<td>241 pounds</td>
</tr>
<tr>
<td>Darex AEA</td>
<td>330 c.c.</td>
</tr>
</tbody>
</table>

A mix of sand and stone concrete was designed using the same cement factor and water-cement ratio as was used for the light-weight mixes. The stone aggregate was a hard durable limestone obtained in the Lehigh Valley. The maximum size of the aggregate was only 1/2", but the fineness modulus was 6.63 which was nearly the same as that of the coarse Solite. The fine aggregate was a washed sand having a fineness modulus of 2.51. Difficulty was experienced in designing a mix with this aggregate which would be as workable as the Solite concrete without changing the cement factor or the water-cement ratio. The mix which was finally selected was not a very practical mix, but was one which best matched the properties of the Solite concrete. The smaller size of the conventional aggregate was the main reason why a workable mix could not be proportioned to give the required strength. The proportions used in the mix were as follows:

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>8 sacks of Type III</td>
</tr>
<tr>
<td>Sand</td>
<td>1625 pounds</td>
</tr>
<tr>
<td>Stone</td>
<td>2345 pounds</td>
</tr>
<tr>
<td>Water</td>
<td>268 pounds</td>
</tr>
</tbody>
</table>
The mixing procedure for the Solite was to mix aggregate and most of the water for 1 minute then the cement was added with the remainder of the water and mixed for 3 minutes. The Darex AEA was added in the mixing water for the air entrained mix. The mixer used for producing the concrete was a Lancirick horizontal countercurrent type mixer having a mixing capacity of 1.5 cubic feet.

A slump of 2 inches was maintained for the plain Solite concrete and the conventional concrete. The slump of the air entrained concrete was 1-1/2 to 2 inches. The most workable mix and the one having the best finishing properties was the air entrained Solite mix. The mix having the poorest workability and poorest finishing properties was the conventional concrete.

2. Construction of Loading Rigs

A means of maintaining a constant sustained load on standard concrete test cylinders was required for the test program. Several types of gravity loading devices were considered, but these were all considered to be too cumbersome. A necessary feature of a loading rig in addition to maintaining a nearly constant load was that it should be possible to obtain stress-strain curves on the concrete cylinders in the loading rig. Figures 1, 2, and 3 show
the loading rig which was used for the tests. The disadvantage of this rig is that the load must be adjusted periodically because the creep of the concrete causes a decrease in the load. However, this rig was easy to construct and easy to manipulate.
Figure 1. Loading Rig

1-1/8" d. Stressteel bars (with standard nut and washer)

7 springs 5-13/16" dia x 8"

Bearing PL. 6-3/4" dia x 1"
Bearing PL. 6-3/4" dia x 1"

Dynamometer

P.L. 20" x 20" x 2"
Figure 2. Loading Rig with Hydraulic Jack in Position

Steel bearing plate

Bakelite gages on both sides

Extra strong steel pipe

Dynamometer

Figure 3.
To load a column of cylinders in the rig a hydraulic jack is connected to the protruding bar on the loading end of the rig (see Figure 2). The jack used is of the type used for post-tensioning bars for prestressed concrete. The springs are compressed as the jack is loaded. When the springs have been compressed the desired amount, the nuts on the bars are tightened so as to move the plates at opposite ends of the column of cylinders together. See sketch of loading rig in Figure 1. After the nuts are tightened equally, the jack is released slowly. When the load has been completely transferred to the cylinders, the actual load on the cylinders is accurately measured by the calibrated dynamometer which forms part of the column being loaded. At any later time, any load lower than the sustained load can be obtained on the cylinders by connecting the jack and transferring part of the load exerted by the springs from the cylinders to the jack by squeezing the springs together. This method was used in changing the loads to obtain stress-strain curves.

The springs used in making the rigs were second-hand freight car springs having a capacity of 16,000 pounds. The springs were calibrated and pairs of springs were selected so that clusters of 7 springs were made up which
were nearly symmetrical and acted as the equivalent of 1 large spring when placed in the rig.

The dynamometer shown in Figure 3 consists of a 10 inch section of 6 inch diameter extra-strong steel pipe. Bakelite SR-4 strain gages were installed as shown on opposite sides of the pipe. The dynamometers were calibrated up to a load of 100,000 pounds using the SR-4 gages wired as a bridge network to measure strains. Loads were measured in the loading rigs from the calibration strain readings. This method of measuring the load was used because the pressure gage on the jack was not accurate. Also there was a loss of load in transferring the load from jack to cylinders which could not be determined except by a load cell in the column of cylinders.

One loading rig was constructed to verify the methods which had been proposed for the test program. It was necessary to determine if the usual procedure for capping concrete cylinders would produce true enough caps for loading a column of 4 separate cylinders. The eccentricity of loading was checked and found to vary from cylinder to cylinder. Spherical bearings were tried in order to improve the alignment, but these did not solve the problem because the jack introduces an eccentricity of its own when under
load and the spherical bearings adjust to this eccentricity. However, when the load is entirely transferred to the cylinders, the eccentricity is different and depends upon the alignment of the cylinders. The spherical bearings cannot readjust because the load on them is too large, and the frictional forces on the bearing surface are large and will not allow rotation of the bearing. The spherical bearings were removed and rubber tile was placed at each joint in the column so that the deformations of the tile would compensate for imperfections in the caps on the cylinders.

The possibility of measuring the sonic modulus of the cylinders under load was considered because of the unknown effects of loading and unloading the cylinders to obtain stress-strain curves. This idea was abandoned because the sonic equipment was not available. Also, some of the difficulties involved in calibrating the sonic apparatus to the loading rigs would probably lead to inaccuracies which might be too large to permit acceptable results. It seems that this method would be feasible if a single column rather than a stack of four cylinders were being loaded. The principle advantage of this method would be that data on both creep and sonic modulus could be taken on the same specimens because there would not be any loading and unloading which affects the creep data.
3. Tests on Instrumentation Systems

It was decided to use two entirely independent instrumentation systems in taking readings on the actual test specimens. Readings were taken mechanically with a gage resembling a Whittemore Gage and by electrical resistance strain gages.

The standard Whittemore Gage was not used because it was felt that a 10 inch gage length would place the plugs too near to the caps on a cylinder 12 inches long. There was concern about readings being affected by end conditions of the cylinders and also the stress concentration around the plugs in the vicinity of the caps. A gage having an 8-1/2 inch gage length consisting of sliding frame and an Ames Dial reading to 1/10,000 inch was used instead of the standard Whittemore Gage. This gage is shown in Figure 4. Plugs similar to the ones used for readings with a Whittemore Gage were cast into the concrete cylinders.
Figure 4. Mechanical Gage Being Read on Creep Specimen

Figure 5. Aluminum Strips on Which SR-4 Strain Gages Are Mounted
The use of Type A-1, SR-4 electrical resistance strain gages was considered to be the best way of taking the data for modulus of elasticity under load. It would also be desirable to have these gages stable enough so that creep and shrinkage data could be taken by them also. Unfortunately, it is virtually impossible to attach SR-4 strain gages to concrete and waterproof them so that they will remain stable over a long period of time. Any method of waterproofing the concrete around the gage would alter the properties of the concrete and make the data uncertain. The possibility of putting the gages on metal which would be cast into the concrete was studied. It was considered undesirable to use an amount of metal which would serve to reinforce the concrete. It was calculated that a small steel bar would be stressed beyond the elastic limit because of its high modulus of elasticity. Gages were placed on a 10 inch long piece of 1/4 inch diameter aluminum tubing and cast in the concrete. Although the aluminum performed satisfactorily, the waterproofing was not dependable. Aluminum is, however, a good material to use because its modulus of elasticity is only about twice that of limestone concrete.

The method finally adopted for attaching SR-4 gages to the concrete was to cast pieces of aluminum channel
11 inches long in the sides of the cylinders. Figure 5 is a sketch of these channels. The nails were added to insure good bond between aluminum and concrete. The gages were put on the exposed aluminum surface after the cylinders were cured. The gages were then waterproofed with shellac. Figure 6 shows the aluminum strips being cast in the concrete cylinders. On several cylinders stress-strain data was taken both by the SR-4 gages on aluminum strips and by the mechanical gage previously described. The plugs for the mechanical gages were placed at 90° on the surface of the cylinders with respect to the aluminum strips (see Figure 6). A typical set of curves which resulted from plotting the data from both mechanical and SR-4 gages is shown in Figure 7. Although the curves did not coincide, they were always parallel over the range of stresses within which they were to be used. The plot of the SR-4 readings sometimes fell above that of the mechanical gage readings and sometimes below, but the difference was never larger than that indicated in Figure 7.
Figure 6. Casting of Limestone Concrete Specimens with Aluminum Strips and Plugs for Mechanical Gage

Figure 8. Buckling of Aluminum Strip at Failure of Concrete Cylinder
Figure 7. Comparison of Stress-Strain Data from SR-4 Gages and Mechanical Gage Taken on the Same Cylinder

Load in Kips

Strain in Millionths
It might be suspected that the aluminum strips would tend to reduce the stresses by acting as reinforcement. The strips were tested on lightweight cylinders which would be low making the modular ratio as large as possible. There was no indication that the aluminum strips were reducing the strains. The aluminum used was a soft aluminum produced by Reynolds Aluminum Company for home repairs and its modulus of elasticity is therefore quite low. The area of 2 aluminum strips in cross section is 0.10 square inches which is 0.36% of the area of a cylinder. This is not enough metal to reinforce the concrete especially since it is aluminum. The strips did reduce the compressive strength of the concrete cylinders 10% to 15% by creating stress concentrations, but the cylinders did not usually fail along the side of the strips. The channels remained bonded up to the ultimate load of the cylinders so that it is believed that the aluminum strips did not modify the properties of the concrete very much. Figure 8 shows a cylinder tested to failure. Notice that the cylinder failed in a direction perpendicular to the strip and easily buckled the strip. Also the strip remained bonded to the concrete except at the point of failure.

In attempting to determine shrinkage and creep of concrete, it greatly simplifies the reduction of the data
if temperature and humidity are controlled throughout the
test. Also the validity of the data may be better under
these conditions. Since it was impossible to control
temperature and humidity in a large enough space for these
tests, the tests were conducted in a room of Fritz Engineering
Laboratory which was air conditioned by the air conditioning
system for the entire building. Although this did not
provide strict temperature and humidity control because
of the variable load on the air conditioning system, the
temperature was maintained in the range of 70° F. to 76° F.
most of the time. The relative humidity varied from 45%
to 60% with only short periods which were outside this
range. Constant records of temperature and humidity were
kept by means of the two recorders shown in Figure 9.
Figure 9. Temperature and Relative Humidity Recorders

Figure 10. Temperature Compensating Cylinder for SR-4 Instrumentation
The usual manner of temperature compensating an SR-4 instrumentation system is to place a gage on a stand-by specimen exactly like the one being tested. This method is suitable for concrete tests of short duration. If this method is used for long time tests, the compensating gage is placed on a specimen which is also shrinking. Therefore this gage is compensating for temperature, humidity, and shrinkage changes. This does not permit the obtaining of suitable data for either shrinkage or creep.

A better method than this is necessary for a long time test. It is known that a concrete specimen kept saturated will not shrink. It may exhibit a slight expansion after a period of time, but the expansion will be small compared to the shrinkage of concrete in air. This suggested a way of making a temperature compensating specimen which would be better than a specimen in air. It can be argued that the curing of a saturated specimen is different from that of a cylinder in air hence the temperature of the concrete is different by the effect of different heats of hydration. This effect tends to lessen with time and reduce the error. Another argument is that a saturated specimen does not respond to temperature changes as rapidly as a specimen in air. This effect was minimized by the
fact that temperature changes would take place gradually because of the air conditioning. A specimen pictured in Figure 10 was made up which had waterproofed gages on aluminum strips cast in the surface of it. The specimen was wrapped in burlap and the burlap was kept saturated by capillary action drawing water from the pan below. The specimen was covered with a loose fitting plastic to reduce surface evaporation which would effect the surface temperatures.

Despite the sources of errors in this compensating arrangement, it was believed that the results would be much better than using a cylinder in air. Plugs were cast into the specimen so that a check on volumes could be made by means of the mechanical gage. Readings by the mechanical gage showed that the volume changes were too small to be measured. Unfortunately, a check on the SR-4 gages showed that there was a drift due to failure of the waterproofing after a period of time.

A temperature compensating gage mounted on steel was then substituted for the saturated specimen. The coefficient of thermal expansion of the steel is 0.0000065 and the thermal coefficient for high strength concrete is very nearly the same. The compensating gage does not compensate
for changes in humidity, but with the degree of control furnished by adjustments on the air conditioning system the error involved is not very large.

4. Final Instrumentation for Test Program

Each of the six loading rigs was provided with a load measuring dynamometer. Each dynamometer was wired into a four pole switch box having a selector switch which permits the reading of any of the dynamometers without changes in wiring. Each dynamometer is self temperature compensated by means of the bridge network by which it is wired. The stability and dependability of this type of load cell is extremely good so that loads up to 100,000 pounds could be measured with an accuracy of plus or minus 500 pounds.

Each of the specimens for obtaining data on shrinkage, creep, and modulus of elasticity under load was provided with the same instrumentation. Two aluminum strips 180° apart were cast in the surface of a cylinder, and paper-base SR-4 gages were glued to each strip with Duco cement and waterproofed with shellac. The SR-4 gages were read with a Type M Baldwin Strain Indicator. Each cylinder was also provided with two pairs of plugs for the measurements by the mechanical gage. Pairs of plugs were cast in the cylinder 180° apart and 90° from the aluminum strips. With this
setup the readings from opposite SR-4 gages could be averaged and compared with the average of the other readings given by the mechanical gage.

Cylinders which were tested on a hydraulic testing machine for obtaining the modulus of elasticity of the concrete were also provided with the same instrumentation.

All SR-4 gages were wired into switch boxes so connected that all gages could be read without changing any wiring. To guard against possible drifting of the strain indicator box, several checks were possible. SR-4 gages of the same gage factor were placed on the sides of the dynamometers and wired into the box to be read with the gages on the test specimens. These readings must remain constant if the loads were not changed. This serves as a check on the SR-4 indicator being used to take readings. Since all gages were read every day, the paper base gages on each dynamometer served to help spot any gage on a test specimen in that rig which might be drifting. Another check on the stability of the SR-4 indicator was an accurate 120 ohm resistor which was wired into a pair of terminals of the switch box and read every time the gages were read.

The paper base gages on the dynamometers were also used to check for any eccentricity in the applied loads. The SR-4 gages on the test specimens were set 90° apart
from the paper base SR-4 gages on the dynamometer so that the eccentricity in perpendicular directions could be checked. It was found that a concentric load on the cylinders could be obtained by proper manipulation of the nuts on the bars of the loading rig.

The stability of the paper base SR-4 gages attached using Duco cement and waterproofed with shellac has been satisfactory thus far in the test. The application of the shellac must be made with caution, however. The shellac must be applied and allowed to dry before any readings are taken because it permits leakage of current to ground while it is still wet. Shellac should never be applied to a gage which is under stress because it temporarily softens the bonding material between the gage and the paper base. The shellac does not seem to have any effect on hardened Duco cement. After the shellac has thoroughly dried, the resistance of the gage to ground is sufficiently high to insure good stability. The shellac seems to be adequate for sealing the Duco cement so that it does not absorb moisture from the atmosphere and relax the bond.

In checking the data from the SR-4 gages against the readings of the mechanical gage, it was found that the data checked very well in some cases but in others it did not check. In the cases where the data did not check, the
readings of the mechanical gage were often completely unreasonable. Therefore, it was suspected that the mechanical gage was not giving reliable readings. The mechanical gage was being read several times after completely removing the gage from the plugs between readings to obtain each recorded data point. Because of the method of taking readings, it did not seem possible that the errors were in reading the gage.

Some explanation of the faults with the mechanical gage were developed after some careful observations. First it was observed that the petroleum jelly which had been put on the surface of the plugs to prevent rust was collecting dirt on the surface, and this dirt was getting into the holes in the plugs. Rust was observed on the surface of some plugs, but the amount was not enough to cause serious errors in readings. The fact that the gage was not temperature compensated did not provide large enough range of possible error to account for the apparent errors in the data. If the plugs were initially cast into the concrete at an angle to a perpendicular to the surface, it was more difficult to obtain reliable readings. These plugs were noted and read with more care after that so that the data from them was good.
The major source of error was finally determined to be in the method of installing the plugs. At the beginning of the test it was decided to cast the plugs in the concrete rather than to attach them to the surface with sealing wax or other bonding material. It was felt that having the plugs in the concrete was safer for long time tests. Therefore the plugs were made 1 inch in length and were cast about 3/4" into the concrete cylinder. The plugs were not attached to the surface of the cylinder. The shrinkage of concrete is known to be greater at the surface especially in the early stages of the shrinkage process. This probably resulted in a rotation of the plugs as well as a movement due to linear shrinkage and creep. The rotation plus stresses induced around the plugs under load is undoubtedly the reason for the erratic results obtained from the readings by this gage.

The plugs for mechanical gage readings must not be set deeply into the concrete. It is recommended that for future tests shallow indentations be made in the surface of the concrete by means of temporary plugs at the time of casting, and that the plugs to be used for readings be set by mortar after forms are removed.

A method of making the plugs and modifying the mechanical gage for this test has been conceived. The
trouble in keeping dirt out of the holes has led to the suggestion that the plugs be made by setting stainless steel ball bearings of suitable diameter into a drilled hole in the plug. The bearing could be set by shrinking or by tapping them lightly to seat them in the holes. The plugs could be made of aluminum to make it easier to install the bearings. The contacts on the gage could be easily modified to provide a spherical or conical cavity to receive the ball bearings on the plugs. Both contact surfaces could be kept clean and this source of error eliminated.

Another modification of the instrumentation which is suggested for future tests is that the column of 4 cylinders be replaced by single columns 6 inches in diameter and 4 feet long cast in Sonotubes. The instrumentation on these columns could be reduced to two SR-4 gages and a few pairs of plugs for readings of the mechanical gage. It is recommended that the SR-4 gages be Bakelite base and carefully waterproofed. The use of the aluminum channels has been quite successful and this method is suggested for mounting the gages. The principle advantage other than simplification of the instrumentation in using a single column is that the stress-strain readings could be taken
as the column was being loaded at a constant rate from the jack. The readings would then compare with modulus of elasticity obtained on a hydraulic testing machine using a standard rate of loading. The four foot columns could be capped in the same manner as the test cylinders.
PART III

RESULTS OF TESTS ON SOLITE CONCRETE

1. Testing Program

The actual test program was initiated using three rich mixes which are typical of mixes being used for prestressed concrete applications. The mixes were designed to give a minimum 28-day compressive strength of 7,000 psi. Type III cement was used to give a high early strength. The 4-day compressive strength of the test specimens was 4,300 psi or higher.

All test specimens were poured on the same day. A total of 102 cylinders were cast consisting of an equal number of cylinders of each of the three mixes. Mix A contained limestone aggregate in the proportions given on page 29; this mix will be referred to as "limestone" concrete. Mix B contained Solite aggregate without any admixture and was proportioned as given on page 26; this mix will be referred to as "plain Solite" concrete. Mix C contained Solite aggregate and entrained air in the proportions given on page 29; this mix will be referred to as "air entrained Solite" concrete. On all graphs specimens are given a designation A, B, or C to show which mix they were cast from. Forty-two of the total of 102 cylinders
were provided with aluminum channels and plugs for the mechanical gage. Two rupture modulus beams were also cast from each mix.

About 6 hours after pouring the forms were carefully removed from the specimens, and the cylinders were placed in the moist room as shown in Figure 11. Fog curing in the moist room was continued for 3 days. During this 3-day period the cylinders were removed from the moist room one at a time to be capped so that all cylinders were capped by the end of the moist curing period. On the third day the cylinders were moved from the moist room to the air conditioned room where the test was to be conducted. At this time paper-base SR-4 gages were put on the aluminum strips, and the Duco cement was allowed to harden for 24 hours before the cylinders were loaded. Cylinders were installed in the loading rigs and held in place by cord (see Figure 4) so that the wiring of the SR-4 gages could be completed. All instrumentation was completed and checked so that the cylinders could be loaded at age 4 days.
Figure 11. Test Specimens Being Cured in the Moist Room

Figure 13. Failure of First Group of Cylinders in Loading Rig B
The cylinders were sorted when removed from the moist room into the following 4 groups:

TEST GROUP I. These cylinders consisted of 18 cylinders of each mix without instrumentation. They were to be stored unloaded in the same room with the other specimens and were to be tested at various times to determine the rate of gain of compressive strength of the 3 concretes. The original proposal called for testing 2 cylinders of each mix for compressive strength at ages 4 days, 7 days, 14 days, 28 days, 90 days, 6 months, 1 year, and 2 years. Extra cylinders were to be used if some of the specimens were found to have strengths that were not consistent with the average being obtained. All cylinders were to be tested at a standard rate of loading of 50 psi per second which is the standard rate of loading given in the A.S.T.M. standard method for testing of concrete cylinders for compressive strength.

TEST GROUP II. This group consisting of two cylinders of each mix with instrumentation were to be stored in the air conditioned room and were to remain unloaded. These cylinders were to provide data on the shrinkage of the three concretes.
TEST GROUP III. This group contained 10 cylinders of limestone concrete, 8 cylinders of plain Solite concrete, and 6 cylinders of air entrained Solite concrete with instrumentation. These cylinders were to be loaded in the 6 loading rigs arranged in the manner shown in Figure 12. These cylinders were to be held under a sustained load applied at age 4 days, and were to provide data on creep and modulus of elasticity under load.
"A" Limestone Concrete  
"B" Plain Solite Concrete  
"C" Air Entrained Concrete

Figure 12. Arrangement of Cylinders in Test Rigs
The cylinders in loading rigs A and B were to be maintained under constant loads of 59 kips and 79 kips respectively for the entire test period and were to provide data on creep at these stresses. A load of 79 kips corresponds to 0.4$f'_c$. This load was chosen because it is the maximum stress recommended by the Bureau of Public Roads\(^{30}\) for the prestressing force after losses. A load of 59 kips which corresponds to a stress of 0.3$f'_c$ was arbitrarily selected as a second value of stress for which the test would provide data. Cylinders of all three mixes were loaded in the same rig so that a valid comparison in the performance of the three concretes could be obtained.

The other four loading rigs were to be used for obtaining values of the modulus of elasticity of the concrete under load by periodically running a stress-strain curve on the cylinders by reducing the loads to nearly zero and recording strains while reloading in increments and taking strain readings. The stress-strain curves thus obtained do not exactly provide a true value of modulus of elasticity which can be compared with the results from static tests on cylinders because rates of loading were not the same; but the results do present a direct comparison of the performance of each type of lightweight concrete to conventional concrete of the same strength under the same load.
TEST GROUP IV. This group included 4 cylinders of each mix with full instrumentation. Two cylinders of each mix were tested at four days to obtain stress-strain curves and compute values of the modulus of elasticity of the three concretes at the time of loading. The remaining cylinders were to be stored in the air conditioned room until reaching an age of 28 days. At this time they would be used for obtaining stress-strain curves and modulus of elasticity.

Some extra cylinders were also stored with the test specimens to be used if needed for supplementary tests. The six rupture modulus beams 6 in. x 6 in. x 36 in. were also stored under the same conditions and were tested for modulus of rupture at age 14 days and 28 days. One beam of each mix was tested at these times. For the 28-day beam tests deflection data was to be taken for computing the modulus of elasticity in bending.

2. Revisions of Testing Program

When the cylinders of Group I were tested for compressive strength at 4 days and 7 days, widely varying values of compressive strength were obtained for the plain Solite and limestone concretes. The cylinders of air entrained Solite were more uniform. Cylinders which had originally been cast for compressive strength tests at
1 year and 2 years were tested at 4 days and 7 days in order to obtain a good average value of the compressive strength. At ages 14 days and 28 days the compressive strength was uniform so that 2 cylinders of each concrete gave a reasonable value for the compressive strength.

The original test program called for loading all test cylinders at an age of 4 days to stresses corresponding to 0.4$f'_c$ and 0.3$f'_c$. Loading rig A was successfully loaded to 59 kips without any trouble. When loading rig B was being loaded in increments up to the load of 79 kips, two of the lightweight cylinders suddenly failed at a load of about 43 kips after about 15 minutes under load. Figure 13 shows loading rig B after the failure of the cylinders.

It was decided to use some of the cylinders which had been set aside for the purpose of determination of the static modulus of elasticity at 28 days to replace the first set of cylinders in rig B. The static modulus of elasticity at 28 days was then found from extra cylinders which had been cast without gages by cementing SR-4 strain gages on the surface of the concrete with Duco cement 2 days prior to testing.

Loading rig B was then loaded again, and this time a lightweight cylinder failed at a load of approximately 40 kips after being under load for 1 minute. The
dynamometer had been recalibrated between failures and found to be reading correctly. In each case the cylinders which failed were of the lightweight concrete. The fractured surface of the cylinder which failed in the second loading of rig B is shown in Figure 14.
Figure 14. Fractured Solite Cylinder from Second Group of Cylinders Loaded in Rig B

Figure 16. Failure of Solite Concrete Cylinders in Column of Three Cylinders
Loading rig D was then loaded in increments to a load of 70 kips. At this load a vertical crack formed in one of the lightweight cylinders. The load was left at 70 kips, and it was noticed that the crack lengthened after about 2 hours but the cylinder remained intact.

After this experience it was decided to allow the concrete cylinders which had not been loaded a longer curing time. Loading rigs C, E, and F were therefore loaded at age 8 days up to a load of only 40 kips or about 1400 psi.

The plan for tests on creep and modulus of elasticity under load was revised to that shown in Figure 15. Loading rig D, which contained cylinders stressed to 2500 psi, was used to provide creep data. Loading rig C was selected for providing creep data at a stress of 1400 psi. This left loading rigs A, E, and F for obtaining data on the modulus of elasticity under load.
Figure 15. Arrangement of Cylinders in Test Rigs

"A" Limestone Concrete
"B" Plain Solite Concrete
"C" Air Entrained Solite Concrete
The type of fracture occurring in the cylinders which failed was a vertical splitting fracture. Examples of this type of fracture are shown in Figures 13, 14 and 19. This type of failure might suggest that the applied load on the cylinders was eccentric. However, in the case of the first cylinders which failed, strain readings had been taken on the cylinders at the failure load and there was essentially no eccentricity. In the case of the second failure, eccentricity could not be checked at the failure load. Little eccentricity was present in the lower load increments.

It was next assumed that the loading of a stack of 4 cylinders as a column was responsible for the failure. Among the points discussed about the manner in which load was applied to the cylinders were:

1. The stack of cylinders is only as strong as the weakest cylinder and therefore not necessarily as strong as any particular cylinder.

2. The possible weakening effect of the aluminum strips in the cylinders.

3. The weakening effect of the end conditions resulting from the number of joints in the column.

4. The possible effect of the flowing action in the rubber tile could reduce the strength of a concrete cylinder.
in a manner similar to the reduction in compressive strength which occurs if the caps of a cylinder are greased before testing.

5. The rate of loading in the rigs was variable. None of these points provided a satisfactory explanation except the possibility that the trouble was caused by loading a stack of cylinders as a column. The reduction in compressive strength caused by the aluminum channels had been determined to be of the order of 10% to 15%. In testing the effectiveness of the rubber tiles in distributing loads on cylinders, individual cylinders had been loaded to 120 kips with rubber tiles at the ends with no evidence of failure. The rate of loading in the loading rigs could not have had much effect on the strength of the cylinders because the load had been released slowly.

In attempting to find the reason for the failure at low loads, a stack of 3 cylinders was first tested in a hydraulic testing machine as shown in Figure 15. The rubber tiles were used without any other means of trying to obtain a concentric load on the cylinders. Each cylinder was first carefully tested up to its ultimate load to determine its approximate strength. The lowest ultimate strength was 145,000 pounds. The testing to the ultimate load undoubtedly destroyed the structure of the concrete so that a second
load test would not reach as high a load as the first. The cylinders were stacked and tested at a rate of loading about the same as that used in the loading rig. The failure load was 88,250 pounds. Two cylinders failed by splitting vertically as in the rigs. This reduction in load was the result of eccentricity, effect of rubber tile, and destruction of the concrete structure caused by tests for ultimate strength. It appeared that the cause of the trouble in the test rig was not present. The three cylinders tested were all lightweight concrete but without instrumentation.

Next a stack of 4 cylinders was tested as shown in Figures 17 and 18. The top two cylinders in the stack were Solite concrete and the bottom two were of conventional concrete one of which had aluminum strips and plugs. The stack was loaded using spherical bearings because it had already been shown that eccentricity was not the cause of failure in the rigs. Also it was observed in testing single cylinders in a hydraulic testing machine for stress-strain curves that the eccentricity present even when a spherical bearing was used was much larger than the eccentricity of loading in the rigs. Figure 18 shows the failure of the column. Figure 19 shows the fractured cylinders. The
same vertical fracture was noted in this stack of cylinders. The failure load on the stack of 4 cylinders was 113,500 pounds, and again it was a lightweight cylinder which failed. The compressive strength of the lightweight cylinders tested on the same day had been 150,000 pounds. This test still did not produce conclusive results.
Figure 17. Loading of Column of Four Cylinders in Hydraulic Testing Machine

Figure 18. Failure of Solite Concrete Cylinders in Column of Four Cylinders
The consistent failure of Solite concrete cylinders suggested that there might be excessive creeping of the concrete at high stresses. To check on this a single cylinder of Solite concrete was placed in the hydraulic testing machine. The cylinder had aluminum strips and plugs cast into it. SR-4 gages were used to measure strains. The cylinder was loaded for periods of 5 minutes at 70 kips increments beginning at 40 kips. There was no evidence of creep until loads above 100 kips were applied. There was evidence of creep under constant loads of 110 kips and 120 kips, but the creep was not excessive. Since these loads exceeded 80% of the ultimate strength of the cylinder, creep at these loads would be expected for any kind of concrete. The cylinder failed after being held at a constant load of 130 kips for 3 minutes. The cylinder did not split vertically but failed as shown in Figure 20. Creep was apparently not the cause of failure in loading rigs, nor were the aluminum strips.
Figure 19. Vertical Fracture of Broken Cylinders from the Column of Four Cylinders

Figure 20. Failure of Solite Concrete Cylinder Held Under Constant Load of 130 Kips
A satisfactory explanation for the failure of lightweight cylinders in the rig was obtained by plotting the stress-strain curves obtained from strain readings of the cylinders which failed at loads below the failure load. Figure 21 shows the stress-strain curves obtained from the first set of cylinders loaded in loading rig B. Curve for limestone concrete is the average of two cylinders. It will be noted that the two cylinders which failed had a modulus of elasticity of about $1.9 \times 10^6$ psi. Figure 22 shows the stress-strain curves for the second group of cylinders loaded in loading rig B. The modulus of elasticity of the cylinder which failed in this instance was only $1.0 \times 10^6$ psi. The lowest value of the modulus of elasticity of any cylinders which had been successfully loaded in loading rigs A and D was $2.8 \times 10^6$ psi. A check of other available values of modulus of elasticity for the lightweight concrete indicates that there were other cylinders having a modulus of elasticity as low as $2.4 \times 10^6$ psi. None were as low as those which failed in loading rig B.
Figure 21. Stress-Strain Curves of First Group of Cylinders Loaded in Rig B - Age 4 Days

Sustained load of 1600 psi applied at 4 days. Cylinder of air entrained Solite and one cylinder of plain Solite exploded after 15 minutes under load.

- Limestone: $E = 2.6 \times 10^6$ psi (Avg. of 2 cyl.)
- Air entrained Solite: $E = 1.9 \times 10^6$ psi
- Plain Solite: $E = 1.9 \times 10^6$ psi
Figure 22. Stress-Strain Curves of Second Group of Cylinders Loaded in Rig B - Age 4 Days

- Limestone: $E = 2.9 \times 10^6$ psi
- Plain Solite: $E = 2.0 \times 10^6$ psi (Avg. of 2 cyl.)
- Exploded at stress of approximately 1400 psi
The reason for the low modulus of elasticity of only a few cylinders must be attributed to poor curing. The equipment in the moist room in which the cylinders were cured was not adequate for maintaining a dense fog throughout the room. The equipment was producing a spray of water which did not permit the maintaining of a fog in all parts of the room. The equipment was producing a spray of water which did not permit the maintaining of a fog in all parts of the room. The room was filled up by the specimens for this test, and apparently some of the specimens in the corners of the room dried out more than the others. It is also possible that during the capping operation some cylinders dried too much and then were replaced in a point in the room where the humidity was not sufficiently high to permit resaturation.

The low modulus of elasticity of the concrete in the cylinders which failed caused the ultimate strain of the concrete to be exceeded even though the load on the cylinders was not very high. It is not possible to state, on the basis of the tests completed to date, a value for the ultimate strain of the Solite concrete. In making compression tests on cylinders of Solite concrete, it was noticed that the ultimate strain does not greatly exceed the strain at ultimate load. For conventional concrete having a 28-day strength of 6000 psi the strain at ultimate load will be about 0.69 times the ultimate strain. This value is
computed from the following formula proposed by Vernon Johnson\(^6\):

\[
B = \frac{1}{1 + (f'c/4000)^2}
\]

In this formula \(B\) is the ultimate strain minus the strain at ultimate load divided by the ultimate strain. The value of \(B\) is therefore 0.308 for concrete with a 28-day strength of 6,000 psi.

Nordby of the University of Colorado also observed that the ultimate strain of his expanded shale concrete was not much higher than the strain at the ultimate load. The vertical splitting type of failure was also reported by him. This low value of the ultimate strain will result in failure at low loads if the modulus of elasticity is low as a result of poor curing. This is more critical in lightweight concrete than in conventional concrete because the modulus of elasticity of the lightweight concrete is lower to begin with.

3. Tests to Determine Compressive Strength

The compressive strengths obtained from testing the cylinders in Group I showed that the rate of gain of the compressive strength for the Solite concrete is the same as that for conventional concrete (see Figure 23).
Figure 23. Rate of Gain of Compressive Strength of Concrete Mixes
The strength of the air entrained Solite concrete was higher than either the plain Solite or the conventional concrete. The reasons for this are first that advantage was taken of the improved workability to decrease the water-cement ratio slightly and second the improved workability increased the effectiveness of the vibration. It was observed in testing cylinders for ultimate load that the air entrained concrete was more uniform as well as being of higher strength. The air entrained concrete did not show as much tendency to fracture suddenly when the ultimate load was reached as did the plain Solite concrete. The cylinders of Group I were weighed at age 28 days to determine the unit weight of the three concretes. The unit weights were as follows:

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional concrete</td>
<td>153.5 lbs./cu.ft.</td>
</tr>
<tr>
<td>Plain Solite concrete</td>
<td>117.8 lbs./cu.ft.</td>
</tr>
<tr>
<td>Air entrained Solite</td>
<td>115.8 lbs./cu.ft.</td>
</tr>
</tbody>
</table>

The saving in dead weight afforded by the use of a concrete mix designed with a cement factor of 8 sacks per yard and a water-cement ratio of 0.356 by weight is 23.3% if plain Solite concrete is used and 24.5% if air entrained Solite is used. These percentages will increase if leaner mixes are used.
4. Shrinkage Measurements

A graph of the shrinkage of the Solite concrete as compared with that of the conventional concrete is shown for the first 28 days of the test in Figure 24. The shrinkage of the air entrained Solite was so nearly the same as the shrinkage of the plain Solite concrete that the two curves coincide. The curves show that for the first 28 days the shrinkage of the Solite concrete is less than that of limestone concrete. This cannot be taken to mean that the Solite concrete will not shrink as much as conventional concrete. It has been pointed out that other investigations have shown that the shrinkage of expanded shale concrete is apt to take place over a longer period of time than for conventional concrete.

It seems reasonable to suppose from these results that the shrinkage of the Solite concrete will not be excessive. The aggregate in the conventional concrete is limestone which tends to produce a concrete of lower shrinkage than most other types of conventional aggregate.
Figure 24. Comparison of Shrinkage of Solite Concrete and Limestone Concrete
5. Creep Tests

Loading rig C was loaded up to a constant load of 40 kips in approximately 10 kip increments. The stress-strain curves for each of the cylinders in the rig is shown in Figure 25. For arrangement of cylinders in the rig see Figure 15. These cylinders were loaded at age of 8 days, and the stress in the cylinders was maintained at 1400 psi by slight adjustments of the rigs.

The elastic strain and the creep plus shrinkage strain for the cylinders loaded in loading rig C is shown in Figures 26 and 27. Each graph compares the strain of a cylinder of plain Solite concrete to the strain of a cylinder of conventional concrete. In Figure 26 the strain of the Solite concrete was greater than the strain of the conventional concrete at 28 days, but this difference is due to the difference in the elastic strains. The creep of the Solite is slightly less than the creep of the conventional concrete. In Figure 27 the strain of the limestone and Solite concrete are equal at 28 days even though the elastic strain for the Solite concrete was larger. It will be noted that the cylinder A2 of limestone concrete had a low modulus of elasticity compared to the average modulus of elasticity for the limestone concrete.
Rig D was loaded up to a constant load of 70 kips. The cylinders were maintained at a stress of 2450 psi after loaded at 4 days. The modulus of elasticity of the cylinders in the rig are shown in Figure 28. Figures 29 and 30 compare the strains of a cylinder of air entrained Solite concrete to the strains of limestone concrete. The creep of cylinders C4 and A2 appear to be equal as shown in Figure 30. The strains of the two cylinders in Figure 29 are equal at 28 days, but again it should be noticed that the modulus of elasticity of cylinder A2 was lower than the average for limestone concrete.
Figure 25. Stress-Strain Curve for Initial Loading of Cylinders in Loading Rig C - Age 8 Days

Cylinder A1
$E = 2.8 \times 10^6 \text{ psi}$

Cylinder A2
$E = 2.4 \times 10^6 \text{ psi}$

Cylinders B3 and B4
$E = 2.4 \times 10^6 \text{ psi}$
Figure 26. Creep Curve for Cylinders in Loading Rig C

- Cylinder B4
- Cylinder A1

Sustained load of 1400 psi applied at 8 days
Figure 27. Creep Curve for Cylinders in Loading Rig C

Sustained load of 1400 psi applied at 8 days

Age of Concrete in Days

Strain in Millionths

Cylinder B3

Cylinder A2
Figure 28. Stress-Strain Curves for Initial Loading of Cylinders in Loading Rig D - Age 4 Days

Stress in psi

Strain in Millionths

Cylinder A2
E = 3.3 x 10^6 psi

Cylinders C3 and C4
E = 2.8 x 10^6 psi

Cylinder A1
E = 2.9 x 10^6 psi
Figure 29. Creep Curves for Cylinders in Loading Rig D

Cylinder A1

Cylinder C3

Sustained Load of 2450 psi applied at 4 days

Age of Concrete in Days

Strain in Millionths

1500
1200
900
600
300
10 20 30 40 50 60 70 80 90 100
Figure 30. Creep Curves for Cylinders in Loading Rig D

- Cylinder C4
- Cylinder A2

Sustained Load of 2450 psi applied at 4 days

Strain in Millionths

Age of Concrete in Days
The data from the loading rigs C and D shows that the creep of Solite concrete is equal to or less than the creep of the conventional concrete, but again the possibility that expanded shale concrete may require a longer time to reach equilibrium must be recalled. The creep of the air entrained Solite porbably does not differ from the creep of the plain Solite concrete.

6. Static Modulus of Elasticity Tests

Some of the cylinders of Group IV were tested at age 4 days to obtain the static modulus of elasticity of the three types of concrete. Figure 31 shows the stress-strain curve for conventional concrete, Figure 32 shows the curve for plain Solite concrete, and Figure 33 shows the curve for air entrained Solite concrete. The modulus of elasticity of the plain Solite concrete is approximately 70% of the modulus of elasticity of the limestone concrete. The air entrained Solite concrete has a modulus of elasticity which is the same as that of the plain Solite concrete. The stress-strain curves from cylinders of Group IV tested at age 28 days are shown in Figures 34, 35 and 36. The modulus of elasticity of the plain Solite concrete appears to be 70% of the modulus of elasticity of the limestone concrete. The modulus of elasticity of the air entrained Solite is
higher than the modulus of elasticity of the plain Solite concrete. This is to be expected since the compressive strength is also higher.

It can be argued that the number of cylinders tested was not sufficient to obtain a good value of the modulus of elasticity of any kind of concrete. However, the values of modulus of elasticity obtained in the loading rigs were also considered in arriving at the value of 70% for the relation of the modulus of elasticity of Solite to the modulus of elasticity of limestone concrete so that the average obtained is fairly good. The curves show that the modulus of elasticity of the lightweight concrete increases at the same rate as that for conventional concrete up to 28 days if the concrete is stored under the same curing conditions and is not loaded.
Figure 31. Stress-Strain Curve for Limestone Concrete - Age 4 Days

\[ E = 4.0 \times 10^6 \text{ psi} \]

\[ E = 3.3 \times 10^6 \text{ psi} \]
Figure 32. Stress-Strain Curve for Plain Solite Concrete - Age 4 Days

\[ E_0 = 2.6 \times 10^6 \text{ psi} \]

\[ E_0 = 2.4 \times 10^6 \text{ psi} \]
Figure 33. Stress-Strain Curve for Air Entrained Solite Concrete - Age 4 Days

\[ E_0 = 2.6 \times 10^6 \text{ psi} \]

\[ E_0 = 2.5 \times 10^6 \text{ psi} \]
Figure 34. Stress-Strain Curve for Limestone Concrete - Age 28 Days

E = 4.1 x 10^6 psi (Avg. of 2 cyl.)
Figure 35. Stress-Strain Curve for Plain Solite Concrete - Age 28 Days

\[ E = 2.8 \times 10^6 \text{ psi} \]
(Avg. of 2 cyl.)
Figure 36. Stress-Strain Curve for Air-Entrained Solite Concrete - Age 28 Days

$E = 3.3 \times 10^6$ psi

$E = 2.9 \times 10^6$ psi
7. Variations of Modulus of Elasticity Under Load

Loading rig A was loaded to a load of 60 kips when the concrete was 4 days old. A stress of 2100 psi was maintained except when the stress-strain curves were being obtained. The change in the tangent modulus of elasticity with time is shown for the cylinders of the three concrete mixes of rig A in Figure 37. The shape of the curve for the limestone concrete is erratic, comparison of the curve with all other curves makes it appear that the data for age 22 days was in error. The stress-strain curve was run twice to check this point and the two readings checked very well so that the data is probably reliable.
Figure 37. Modulus of Elasticity Data from Loading Rig A

- Limestone
- Plain Solite (Avg. of 2 cyl.)
- Air Entrained Solite

Sustained load of 2100 psi applied at 4 days

Modulus of Elasticity in Million psi

Age of Concrete in Days
Neglecting the point on the curve for the limestone concrete at 22 days, the rate of increase of the modulus of elasticity under load is about the same for Solite and limestone concretes. It is impossible to determine from the curves if there is any difference in the rate of increase of modulus of elasticity between the plain Solite concrete and the air entrained Solite concrete.

Loading rig C was loaded to a load of 40 kips when the concrete was 8 days old. A stress of 1400 psi was maintained except when taking stress-strain data. The curves of Figure 38 show the changes in the tangent modulus of elasticity for the cylinders of rig C. The curves for the limestone concrete and plain Solite concretes show the same variations, but the modulus of elasticity of the limestone concrete has increased proportionally more than the modulus of elasticity of the plain Solite concrete. In Figure 39, which compares the modulus of elasticity of air entrained Solite to the modulus of elasticity of limestone concrete under a sustained stress of 1400 psi, the curves are of the same shape and the modulus of elasticity of both concretes has increased by about the same percentage in the 28 day period.
Figure 38. Modulus of Elasticity Data from Loading Rig E

Sustained load of 1400 psi applied at 8 days

Note: Curves are avg. of 2 cyl.

- - Limestone
- - Plain Solite

Age of Concrete in Days

Modulus of Elasticity in Million psi

10
8
6
4
2
0

10 20 30 40 50 60 70 80 90 100
Figure 39. Modulus of Elasticity Data from Loading Rig F

Sustained load of 1400 psi applied at 8 days

Note: Curves are avg. of 2 cyl.

- Limestone
- Air Entrained Solite

Age of Concrete in Days

Modulus of Elasticity in Million psi
The percentages of the increase in modulus of elasticity between the time of initial loading and 28 days is as follows:

Loading Rig A (Sustained stress of 2100 psi)

<table>
<thead>
<tr>
<th>Concrete Type</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limestone concrete</td>
<td>38.3%</td>
</tr>
<tr>
<td>Plain Solite concrete</td>
<td>27.9%</td>
</tr>
<tr>
<td>Air entrained Solite concrete</td>
<td>30.8%</td>
</tr>
</tbody>
</table>

Loading Rig E (Sustained stress of 1400 psi)

<table>
<thead>
<tr>
<th>Concrete Type</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limestone concrete</td>
<td>40.6%</td>
</tr>
<tr>
<td>Plain Solite concrete</td>
<td>12.5%</td>
</tr>
</tbody>
</table>

Loading Rig F (Sustained stress of 1400 psi)

<table>
<thead>
<tr>
<th>Concrete Type</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limestone concrete</td>
<td>29.4%</td>
</tr>
<tr>
<td>Air entrained Solite concrete</td>
<td>20.8%</td>
</tr>
</tbody>
</table>

The amount of increase in modulus of elasticity during this period was always greater for the limestone concrete than for either of the lightweight concretes. The percentage of increase in modulus of elasticity is apparently affected by the magnitude of the initial modulus of elasticity. The cylinders of Solite concrete in rig A have a good percentage increase even though they were under a higher sustained load than the ones in the other two rigs. These two cylinders also had higher moduli of elasticity when they were first loaded. It would appear from this that the curing of the cylinders will have an important bearing on the performance of the concrete under load. If Solite concrete were cured more carefully or for a longer period of time, its performance under
load would undoubtedly be greatly improved. It is not known whether or not careless curing is more critical for expanded shale concrete than for conventional concrete. This is a question which deserves investigations. Since the modulus of elasticity of the lightweight concrete is going to be lower than that of conventional concrete, it seems reasonable to assume that curing will be more critical. An accelerated curing method such as steam curing might be useful. Steam curing for lightweight building blocks has been successfully used by block manufacturers to reduce shrinkage and creep and to give high early strength properties to the blocks.

8. Rupture Modulus Tests

Two beams of each mix were tested to determine the rupture modulus of the three concretes. The beams were moist cured for 3 days and then stored under the same conditions as the cylinders until tested. One beam of each mix was tested at the age of 14 days. The comparison of the rupture modulus of the three beams is as follows:

<table>
<thead>
<tr>
<th>Beam</th>
<th>Rupture Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limestone concrete</td>
<td>570 psi</td>
</tr>
<tr>
<td>Plain Solite</td>
<td>436 psi</td>
</tr>
<tr>
<td>Air entrained Solite</td>
<td>390 psi</td>
</tr>
</tbody>
</table>
The same tests were repeated with the remaining three beams at age of 28 days. The results were as follows:

<table>
<thead>
<tr>
<th>Beam</th>
<th>Rupture Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limestone concrete</td>
<td>652 psi</td>
</tr>
<tr>
<td>Plain Solite</td>
<td>472 psi</td>
</tr>
<tr>
<td>Air entrained Solite</td>
<td>390 psi</td>
</tr>
</tbody>
</table>

From these tests it appears as though the tensile strength of the lightweight concrete is being reduced by the lower strength of the Solite aggregate in tension. The air entrained Solite concrete has a lower rupture modulus than the plain Solite concrete. The number of tests was not large enough to be conclusive, but tests do point out a weakness of the Solite concrete which should be investigated further. The modulus of elasticity computed from deflections of the rupture modulus beams could not be correlated with values obtained from test cylinders. Because of the small number of beams tested the values of modulus of elasticity in bending will not be discussed until more results are available.

9. Solite Concrete and Prestressing

The information obtained from this test does not supplement previous test results to an extent that the performance of Solite concrete in prestressed structures can be discussed conclusively. The following discussion should
therefore be considered in the light of the necessity for more research to verify the assumptions used. A pretensioned beam shown in cross section on page 109 was designed using ordinary dense concrete having a unit weight of about 150 pounds per cubic foot and assuming a loss of prestress of 20%. A summary of the calculations for this beam is given on page 110.

The same section was used in calculations for a similar beam of 70 foot span of Solite concrete. A summary of the calculations for this beam is given on page 111. The unit weight of the Solite concrete was taken as 76.7% of that in the previous calculation. The difference in the dead load moment and in the total moment is considerable. The amount of steel required to carry the same live load moment is 43 strands compared to 47 strands for the beam of heavy concrete. This means a saving of 8-1/2% in the amount of steel required for the beam of lightweight concrete if designed for the same span.
PRETENSIONED CONCRETE BEAM SECTION

Section properties

Area $558 \text{ in}^2$
Moment $9232 \text{ in}^3$
$Y_b$ $16.53''$
$Y_t$ $16.47''$
$I_c$ $78900 \text{ in}^4$
$S_b$ $4790 \text{ in}^3$
$S_b$ $4770 \text{ in}^2$
DESIGN
FOR

Loading: H20-S16
Span length: 70 ft
Wearing surface weight: 90 *ft
Beam concrete used: Dense
weight: 580 *ft
28 days strength: 6,000 *in^2
Allowable steel stress: 150,000 *in^2
Pretension loss: 20% Assumed

Design moments

Live load: 3,550,000
Impact: 920,000
Beam: 4,270,000
Surface: 662,000

M = 9,384,000 in-lb

Prestressing

emax: 12.68"
Steel area 47-3/8 cables: 3.76"
Initial prestressing force: 564,000#
Final prestressing force: 451,200#

Stress summary (ksi)

<table>
<thead>
<tr>
<th></th>
<th>P0inal</th>
<th>Pfinal</th>
<th>Delerim</th>
<th>Bsurface</th>
<th>Lstres</th>
<th>3 when</th>
<th>3 when</th>
<th>3 when</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOP</td>
<td>-480</td>
<td>-384</td>
<td>+832</td>
<td>+138</td>
<td>+230</td>
<td>+412</td>
<td>+509</td>
<td>+1517</td>
</tr>
<tr>
<td>BOTTOM</td>
<td>+2500</td>
<td>+2000</td>
<td>-834</td>
<td>-139</td>
<td>-334</td>
<td>+1606</td>
<td>+1106</td>
<td>+33</td>
</tr>
</tbody>
</table>

Ultimate moment: 25,100,000 in-lb
Safety factor: 2.68
Principal tensile stress at centroid over support: -60 psi
Shearing stress 8" from top over support: -59 psi
Principal tensile stress @ 2.5x ultimate load at 8" from top over support: -357 psi
DESIGN FOR

Loading: H20.516
Span length: 70 ft
Wearing surface weight: 90 #/ft
Beam concrete used: Solite
weight: 445 #/ft
28 days strength: 6000 psi
Allowable steel stress: 150,000 psi
Pretension loss: 25% Assumed

Design moments

<table>
<thead>
<tr>
<th>Live load</th>
<th>Impact</th>
<th>Beam</th>
<th>Surface</th>
</tr>
</thead>
<tbody>
<tr>
<td>3,550,000</td>
<td>902,000</td>
<td>3,280,000</td>
<td>662,000</td>
</tr>
</tbody>
</table>

Mₚ: 8,394,000 in.lbf

Prestressing

emax: 13.34"
Steel area: 43.38 cables: 3.44"
Initial prestressing force: 516,000#
Final prestressing force: 387,000#

Stress summary (psi)

<table>
<thead>
<tr>
<th></th>
<th>P Initial</th>
<th>P Final</th>
<th>DL Beam</th>
<th>DL Surface</th>
<th>LL+Imp</th>
<th>L+I+O</th>
<th>L+O+I+O</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOP</td>
<td>-515</td>
<td>-386</td>
<td>+685</td>
<td>+438</td>
<td>+930</td>
<td>+170</td>
<td>+299</td>
</tr>
<tr>
<td>BOTTOM</td>
<td>+2370</td>
<td>+1775</td>
<td>-688</td>
<td>-139</td>
<td>-934</td>
<td>+1682</td>
<td>+1087</td>
</tr>
</tbody>
</table>

Ultimate moment: 23,100,000 in.lbf

Safety factor: 2.76

Principal tensile stress at centroid over support: -36 psi
Shearing stress 8" from top over support: -78 psi
Principal tensile stress @ 2.5 Ultimate load at 8" from top over support: -315 psi
DESIGN FOR

Loading
Span length
Wearing surface weight
Beam concrete used
weight
28 days strength
Allowable steel stress
Pretension loss

H20: 516
75 ft
90 lb/ft
Solite
4.45 lb/ft
6000 lb/in²
150,000 lb/in²
25% Assumed

Design moments

Live load
Impact
Beam
Surface

3,870,000
966,000
3,750,000
758,000

Mf 9,344,000 in·lb

Prestressing

\[ e_{max} \]
\[ 12.77'' \]

Steel area 43-3/8

3.92

Initial prestressing force

589,000 lb

Final prestressing force

442,000 lb

Stress summary (psi)

\[
\begin{array}{ccccccccc}
\text{T OP} & \text{I n i t i a l } & \text{F i n a l } & \text{B e a m} & \text{S u r f a c e} & \text{w h e e l} & \text{I n i t i a l } & \text{F i n a l } & \text{F i n a l} \\
-520 & -390 & 785 & 158 & 912 & 265 & 385 & 1471 \\
\text{B O T T O M} & 1260 & 7200 & -787 & -159 & -920 & +1833 & +1213 & +134 \\
\end{array}
\]

Ultimate moment

26,100,000 in·lb

Safety factor

2.79

Principal tensile stress at centroid over support

-33 psi

Shearing stress 8'' from top over support

-73 psi

Principal tensile stress @ 2.5 Ultimate load at 8'' from top over support

-311 psi
The loss of prestress assumed for the lightweight concrete was 25% or 5% higher than that assumed for the beam of dense concrete. From tests of post-tensioned beams, Koebel recommended 25% loss of prestress. On the basis of this a loss of greater than 25% should be assumed for a pretensioned beam. The beams tested by Koebel were of Haydite concrete which may have different properties than Solite. If one considers the properties of Solite on the basis of the very meager information furnished by our tests, it would appear that the losses in prestress due to shrinkage and creep may be considered equal to the losses in conventional concrete. This means that the difference in losses will be the difference in the loss of prestress due to elastic shortening. These elastic losses can be calculated as follows:\(^{10}\):

\[
p = \frac{n F_0}{A_t}
\]

Where \(p\) is the loss of prestress due to elastic shortening, \(n\) is the modular ratio, \(F_0\) is the initial prestress, and \(A_t\) is the area of the transformed section. Since the amount of steel is small, the area of concrete is a good approximation for \(A_t\).
For the beam of conventional concrete, the loss of prestress is:
\[
p = \frac{6(564,000)}{558} = 6,060 \text{ pounds}
\]
\[
\frac{6,060(100)}{564,000} = 1.08\% \text{ loss of prestress}
\]

For the same beam designed using Solite concrete, the percentage loss of prestress due to elastic shortening will be:
\[
p = \frac{8.57(516,000)}{558} = 8,000 \text{ pounds}
\]
\[
\frac{8,000(100)}{516,000} = 1.55\% \text{ loss of prestress}
\]

These calculations are made on the assumption that the modulus of elasticity of the steel will be 30 x 10^6 psi, the modulus of elasticity of the conventional concrete will be 5 x 10^6 psi, and that the Solite concrete will have a modulus of elasticity of 70\% of the value for conventional concrete.

It will be noticed that the amount of the losses resulting from elastic shortening are very small. Therefore the amount of extra loss of prestress assumed for the Solite concrete need not be increased much to take into account the additional losses which result from the lower initial modulus of elasticity of the Solite concrete. The lower prestressing force required by the Solite concrete beam partly compensates for the lower modulus of elasticity.
in the calculation for the loss of prestress due to elastic shortening only.

The extra 4.5% in the assumption is to take into account any poorer performance of the Solite concrete under load.

In addition to the saving in steel with the lightweight beam, there is a saving in dead weight of about 23%. The factor of safety for ultimate moment is slightly higher than for the heavier beam of conventional concrete.

The lightweight concrete was used in designing a beam for a span of 75 feet using the same cross section which was used for the 70 foot span. A summary of the calculations for this beam is given on page 112. The dead weight of the 75 foot beam made with lightweight concrete is 33,300 pounds compared to a weight of 40,600 pounds for the 70 foot beam of conventional concrete. This is a saving in weight of 18%. The stresses in the 75 foot beam are satisfactory and the safety factor for ultimate moment is higher than for either of the other beams. The additional 5 feet of span is achieved by using 49 strands of reinforcing as compared to 47 strands for the 70 foot span beam of conventional concrete. This is an increase of 4.3% in the amount of steel.
It will be noticed that the actual ultimate moment of the lightweight beams is smaller. Because of the saving in the dead weight of the beams, the live load capacity is higher and the maximum compressive stress in the top fiber at midspan under full live load is lower than in the beam of conventional concrete. The tension stress in the top fiber at the ends of the three beams is about the same. Since the lightweight concrete is slightly weaker in tension than the conventional concrete, this might require more mild steel reinforcement at the ends of the lightweight beams, but on a percentage basis this would not be large. It is difficult to compare the deflections of the beams without any information from actual beam tests. It seems that the additional deflection of the lightweight beams will not be excessive and will probably not exceed 130% of the deflection of the beam made with heavier concrete. The initial camber of the lightweight beams will also be larger, but the camber should not be excessive.

The advantages of using Solite concrete becomes greater as the length of the span is increased. For beams up to a span of about 50 feet, the advantage of lightweight concrete will be only a saving in dead weight. The most important disadvantage of the lightweight concrete results from a lower modulus of elasticity. This may dictate the use of
more carefully controlled curing condition or the release of prestress at a later time if lightweight concrete is used in place of conventional concrete.
CONCLUSIONS AND RECOMMENDATIONS

The following conclusions are based on the limited amount of data available from the first tests of Solite concrete at Lehigh University.

1. For rich mixes using 8 sacks of cement per cubic yard of concrete, the use of Solite concrete in place of conventional concrete permits a saving in dead weight of about 23%.

2. Solite concrete with 28-day compressive strengths up to 8,000 psi can be produced with satisfactory workability, and mixes can be designed based on the water-cement ratio method. The rate of gain of compressive strength does not differ from that of conventional concrete.

3. The use of air entraining in rich mixes of Solite concrete is beneficial and permits a reduction in the water-cement ratio so that there is no reduction in the compressive strength of the air entrained concrete.

4. The Solite concrete shrinks less than limestone concrete during the period up to an age of 28 days.

5. The Solite concrete creepes slightly less than the limestone concrete of the same strength under the same loading conditions during the first 28 days under load.
6. The modulus of elasticity of the Solite concrete with or without air entraining will be about 70% of the modulus of elasticity of conventional concrete. The rate of increase of the modulus of elasticity of Solite concrete is about the same as that of limestone concrete.

7. The modulus of elasticity of the Solite concrete increases at a slightly slower rate than that of the conventional concrete when both are under the same sustained stress.

8. It may be necessary to cure high strength Solite concrete more carefully than ordinary dense concrete if it is going to be loaded at an early stage because of the lower modulus of elasticity and apparent lower ultimate strain safety factor.

9. The rupture modulus of Solite concrete is lower than that of conventional concrete having the same compressive strength.

10. The properties of Solite concrete seem to be such that the use of this concrete in prestressed concrete design is possible.

The present tests should be continued until the test data shows that all three concretes have reached about 90% equilibrium. There is need for more data over a longer
period of time to fortify the conclusions given in this report. It is recommended that at a later date the stacks of 4 cylinders now under test be replaced by single columns 4 feet in length to check the effect of some of the assumptions which were made regarding the action of stacks of cylinders.

An investigation of special curing procedures such as steam curing and vacuum curing which are used to give high early strength should be undertaken particularly to determine if they produce a lightweight concrete with a high early modulus of elasticity.

Some of the questions which arise in using lightweight concrete in prestressed concrete members can be satisfactorily answered only if prestressed members made with lightweight concrete are tested. The quality of the Solite concrete now under test indicates that it is of high enough quality to warrant such tests. The saving in dead weight and steel on long span members which is possible, if it is demonstrated that lightweight concrete prestressed beams can be successfully constructed, could increase the length of spans over which prestressed concrete can compete with steel. The comparative cost of lightweight aggregate and conventional aggregate is a factor which greatly influences the economy of lightweight concrete members and is a problem which must be investigated locally.
REFERENCES


15. Nordby, G. M., "A Study of Steel Strand for Prestressing Concrete Beams of Expanded Shale and Conventional Aggregate", *Engineering Experiment Station, University of Colorado*.


