Expanded Shale Aggregate in Structural Concrete

By
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Bulletin No. 30
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Engineering Experiment Station
Oregon State System of Higher Education
Oregon State College
Corvallis

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FOREWORD AND ACKNOWLEDGMENTS

The experimental work reported in this bulletin was performed by Mr. D. D. Ritchie as a project of the Engineering Experiment Station under the general supervision of S. H. Graf, Director. The investigation formed the subject of Mr. Ritchie's thesis for the degree of Master of Science in civil engineering, which degree was awarded June 1951. The work was made possible by a grant of funds from the Empire Building Material Company of Portland, Oregon, to cover the cost of the assistantship and materials received. Appreciation is expressed to the Company not only for financial support but also for the friendly cooperation and valuable advice of Mr. Frank Spangler, President and General Manager.

Acknowledgment is also made of the assistance and advice of Professors G. W. Holcomb and I. F. Waterman of the Department of Civil Engineering, and Professors C. E. Thomas and C. O. Heath of the Department of Mechanical Engineering.

Most of the tests for the program were made on the light-weight expanded shale aggregate produced by the Empire Building Material Company under the name of "Lite-Rock." For comparison, another similar expanded shale aggregate, purchased from a plant in California, and commercially produced gravel and sand from the Portland market were utilized. Thermally expanded shale light-weight aggregates, as well as light-weight volcanic materials, and ordinary gravel and sand aggregates are extensively used in molded products such as blocks and pipe. In these machine-molded products comparatively dry mixes are used. The work here reported does not cover these dry-mixed products, but is limited to the general application of aggregates to structural concrete as ordinarily mixed and placed.

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Expanded Shale Aggregate in Structural Concrete

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I. INTRODUCTION

1. **Applications of light-weight concrete.** The use of light-weight concrete is not new, it having been employed in the early days of the Roman Empire when pumice was used as a component of temple roof slabs. Today we have rediscovered the practice and many types of light-weight concrete are in use. Probably the most notable example is the placing of "Gravelite" light-weight-aggregate concrete in the upper deck of the San Francisco-Oakland Bay Bridge where a $3,000,000 saving was attributed to the reduction of dead load. Another instance of interest was the addition of six floors to the Argyl Building in Kansas City, Missouri, by using "Haydite," an expanded shale aggregate, where only four floors had been planned with heavy concrete. In Cleveland, the original design of a building was changed by the addition of four mezzanines without enlarging the foundations.

At the time of writing, a building is under construction (since completed) in Portland, Oregon, (Figure 1) where "Lite-Rock" aggregate concrete, which is to be the subject of this paper, is being used. Here a floor 130 feet in clear-span width is achieved by light-weight concrete trusses.

2. **Need for design information.** With the expanding use of light-weight aggregate concrete a demand arises for information descriptive of its behavior. Architects, engineers, contractors, and builders, desiring to use light-weight concrete, require reliable design data as well as a knowledge of characteristics which might govern the choice of material for a particular need.

Existing building codes and regulations for natural aggregates are not applicable to light-weight concrete. Recognition of this fact has resulted in the publication, "Light-weight Aggregate Concretes," (1) recently issued by the Housing and Home Finance Agency. This publication shows not only that these aggregates differ from sand

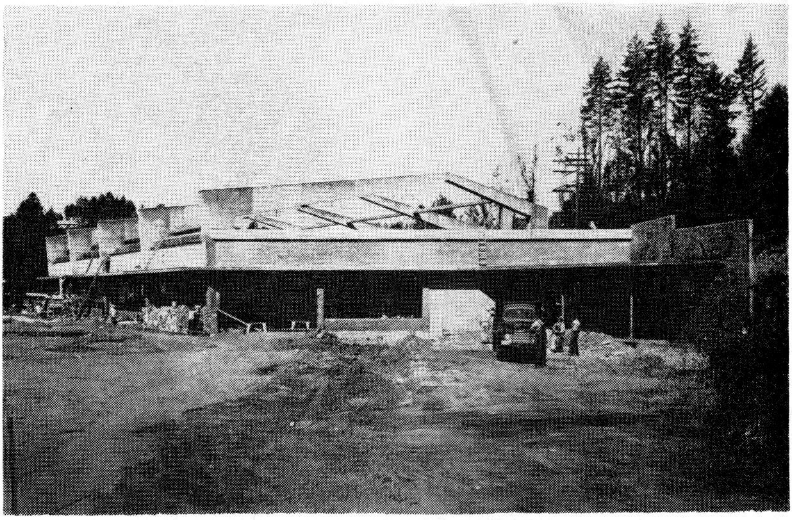

Figure 1. Construction photograph of Fred Meyer Burlingame Shopping Center, Portland, Oregon, with trusses of light-weight concrete.

and gravel, but that wide variations may be expected between different types of light-weight aggregate and that each particular aggregate requires individual study.

It was with the object of obtaining information relating to such a particular aggregate, "Lite-Rock," (a trade name) that the present investigation was inaugurated.

3. **Lite-Rock.** Lite-Rock is the material produced by crushing and burning a certain shale, mined near Banks, Oregon. The burning is accomplished in a rotary kiln at temperatures in excess of 2,200 F. At these high temperatures melting begins and gases are evolved causing expansion of the softened shale by formation of innumerable cells. The outer surface becomes completely melted and upon cooling forms a coating over the inner cellular structure.

In the past this expanded material has been recrushed when discharged from the kiln. This produces a rather harsh aggregate and one which has the cellular structure exposed to invite absorption. During the course of this project, however, it was learned that a considerable portion of the kiln output could be obtained in such sizes that further crushing was unnecessary. The testing program was carried out using this uncrushed material. Preliminary tests on the crushed expanded shale are dealt with briefly in Section II.

Table 1. Outline of Principal Tests

Test series	Specimens: Size	Curing	Number of specimens: Mix A	Mix B	Mix C	Mix D	Mix C_r	Mix D_r	Mix E	Mix G	Mix H
Compressive strength (Mod of elasticity)											
Test 1	4" x 8" cyl	7 day moist	3	3	3	3	3	3	3	3	3
Test 2	4" x 8" cyl	7 day moist, 21 day air	3	3	3	3	3	3	3	3	3
Test 3	4" x 8" cyl	28 day moist	3	3	3	3	3	3	3	3	3
Test 4	6" x 12" cyl	28 day moist	3	3	3	3	3	3	3	3	3
Test 5	4" x 8" cyl	7 day moist, 83 day air	3	3	3	3	....	....	3	3	3
Flexure—Test 6; Sonic modulus—Test 6_s	6" x 6" x 36"	28 day moist	1	1	1	1	1	1	1	1	1
Bond—Test 7	8" x 8" cyl	7 day moist, 21 day air	3	3	3	3	3	3	3	3	3
Dorry abrasion—Test 8	2" x 4" cyl	7 day moist, 21 day air	1	1	....	....	1	1	1	1	1
Absorption—Test 9	4" x 8" cyl	7 day moist, 21 day air	3	3	....	....	3	3	3	3	3
Shrinkage—Test 10	3" x 3" x 11"	7 day moist, 21 day air	3	3	....	....	3	3	3	3	3

Nine mixes were used as follows:

Aggregate: A, B, C, D, C_r, D_r, E—Lite-Rock; G—gravel; H—expanded shale No. 2. All dry batched. Maximum size: A, B—¾"; C, D, E, H—⅜"; C_r, D_r—¼"; G—1".

Cement factor: sk cu yd: A—3.7; B—5.4; C—6.9; D—9.2; C_r—6.9; D_r—8.8; E—7.1; G—4.8; H—6.9.

Dispersing agent: ½ lb per sack cement in all but mix E.

Water: Sufficient to provide good workability.

4. **Outline and scope of work.** The investigation reported here consists primarily of tests on Lite-Rock expanded shale aggregate concrete. For comparison, similar but limited tests were made using two other aggregates, natural sand and gravel, and a second expanded shale. Sections through the light-weight concretes are shown in Figures 2 and 3. The materials used in the tests are described in Section III, and their proportioning and mixing in Section IV. The concrete tests are outlined in Table 1, described in Section V, and furnish material for the discussion and design data taken up in Sections VI and VII.

The testing program was arranged to facilitate comparison with the extensive work done on light-weight aggregate concretes by the Bureau of Reclamation and the National Bureau of Standards which is reported in "Light-weight Aggregate Concretes" (1). Cement factors were chosen in the neighborhood of 3, 5, 7, and 9 sacks per cubic yard to correspond with the government tests. In the comparison tests, cement factors of approximately seven for expanded shale No. 2 and five for the sand and gravel were used. The mixes are taken up in detail in Section IV.

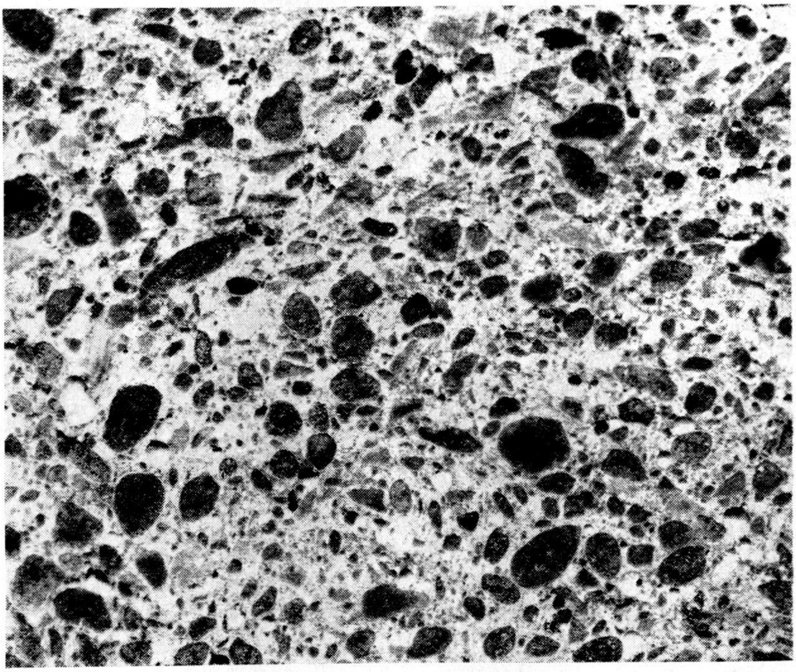

Figure 2. Section through Lite-Rock concrete (actual size).

Figure 3. Section through expanded shale No. 2 concrete (actual size).

II. PRELIMINARY TESTS ON BEAMS USING CRUSHED LITE-ROCK CONCRETE

1. **General.** The tests on crushed Lite-Rock beams are included here because of their usefulness in supporting design theory which is set forth in Section VII. These tests were to be a part of the program as originally planned and are termed "preliminary" because of the subsequent change to uncrushed aggregate. The tests are illustrated in Figures 4 to 7.

The beams were poured and tested by senior students in civil engineering enrolled in the Structural Materials Laboratory course. Five beams were tested, two of crushed Lite-Rock aggregate and three of sand and gravel. Comparison tests were made between Lite-Rock and gravel concrete beams with and without stirrups, and a fifth beam of gravel concrete was tested which was provided with both tension and compression steel.

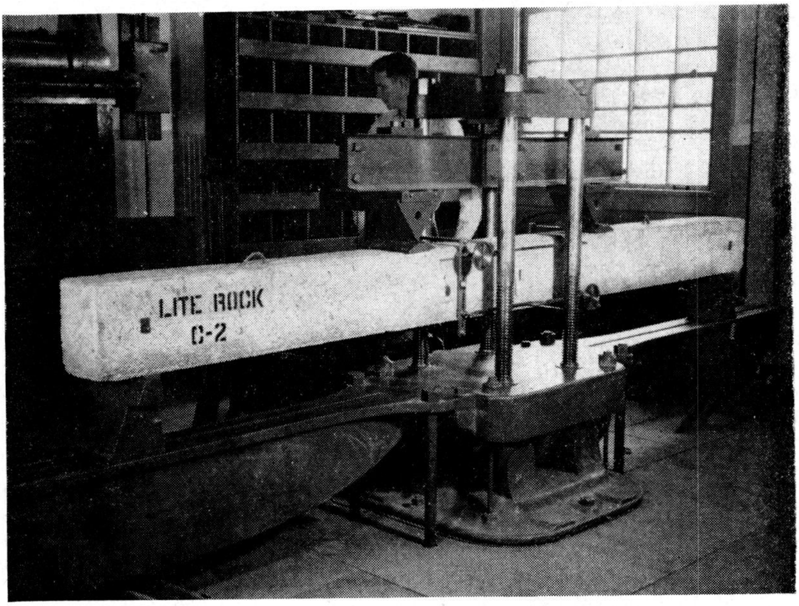

Figure 4. Beam test on crushed Lite-Rock concrete.

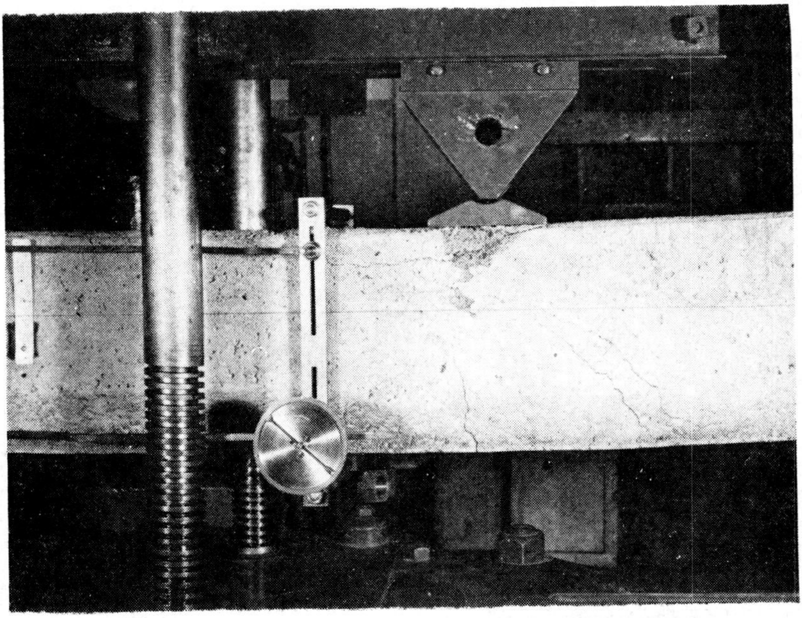

Figure 5. Failure of crushed Lite-Rock beam due to tension in steel.

2. **Mixing.** To avoid drying out of the mix, the crushed Lite-Rock aggregate was soaked in the mixing water for about five minutes prior to mixing. A dispersing agent ("Pozzolith"), dissolved in a portion of the mixing water, was added to the mixture. Best results were obtained by withholding the dispersing agent until after the soaking period.

The capacity of the mixer was found to be reduced about one-third by the light-weight aggregate, and another problem was encountered in the tendency of the fine aggregate to stick to the sides of the mixer. Apart from this, the beams were poured without difficulty and with little departure from ordinary methods.

3. **Diagonal tension test.** In the beam test without stirrups (Figure 6), the Lite-Rock beam attained slightly greater load than the gravel beam, but less than would be expected considering a higher compressive strength. This deficiency in diagonal tension resistance for crushed Lite-Rock concrete was in accord with lower values for modulus of rupture as found on plain concrete beams. No such deficiency exists in the uncrushed Lite-Rock concrete as will be seen in Part V of this paper.

4. **Beams with web reinforcing.** The two beams with stirrups (Figure 7) failed at loads approximately proportional to their compressive strengths. The ultimate loads are not of great significance, however, as the failure in both cases was due to tension in the steel. The most interesting comparison is that of the relative stresses in Lite-Rock and gravel concrete beams for equal loads. This will be discussed in Part VII where design of Lite-Rock concrete is considered.

III. CONCRETE MATERIALS

1. **General.** One lot of ordinary portland cement ("Oregon" brand) was used for all mixes. The admixture, which was a dispersing agent rather than an air entraining agent, was one recommended by the manufacturers of Lite-Rock aggregate. The steel used in the bond tests was of structural grade.

2. **Description of the aggregates.** Coarse and fine Lite-Rock aggregates are pictured in Figures 8 and 9. These are composed of expanded shale particles as they are discharged from the kiln, without recrushing. Each particle, having been heated to the point of fusion, retains on its surface a coating of melted shale. This product is not as smooth as natural gravel, but much less harsh

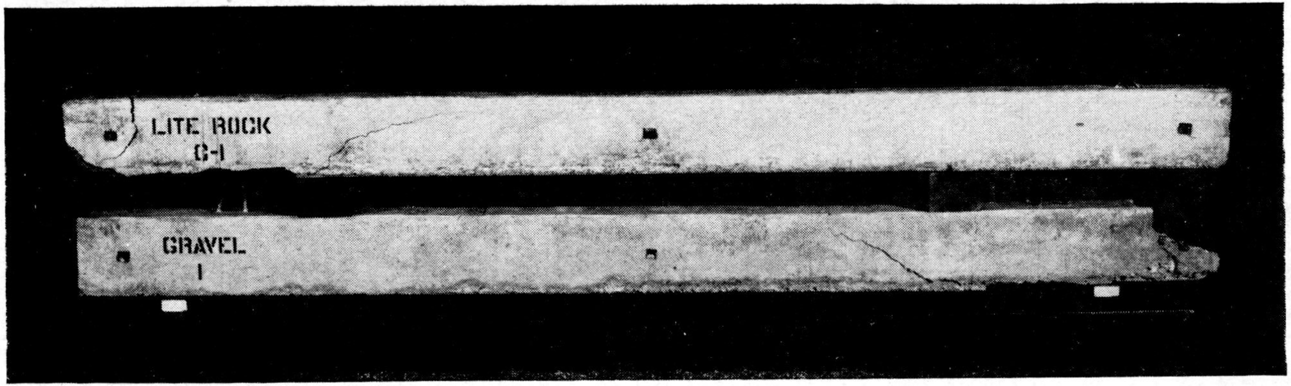

Figure 6. Beams without stirrups, after test.

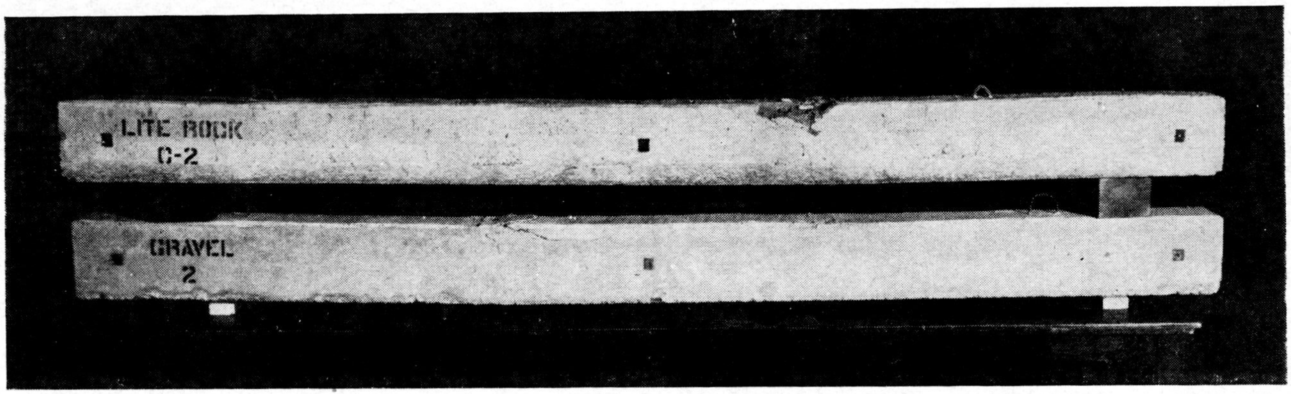

Figure 7. Beams with stirrups, after test.

than a crushed stone, or a shale which has been crushed after expansion. The glaze coating also provides protection against absorption which is materially reduced from that for the crushed aggregate. Another advantage is that less surface is exposed to cover with cement paste than with an aggregate having an exposed cellular structure like that of the crushed material.

Figure 8. Coarse Lite-Rock aggregate (actual size).

The expanded shale used for comparison, and termed "Expanded Shale No. 2" in this paper, was shipped in from California. The coarse and fine aggregates are shown in Figures 10 and 11. This aggregate was more harsh than Lite-Rock, having been partly recrushed as shown in the photograph. However, much of it was coated and it differed from Lite-Rock principally by its greater weight.

Figure 9. Fine Lite-Rock aggregate (actual size).

Columbia River sand and gravel were obtained from a commercial source in Portland, Oregon, to represent the aggregate with which Lite-Rock would normally compete.

3. **Sieve analysis.** The Lite-Rock aggregate was shipped from the plant in sacks and was used as received except where it was necessary to remove sizes larger than desired. Sieve analyses were taken on representative samples from each mix and are shown in Table 2 along with those for the two comparison aggregates. Separation at the plant was not exact and it will be noticed that some of the fine aggregate was retained on a No. 4 sieve. This needs to be considered when making a study of proportions used in the concrete mixes.

4. **Unit weight.** Unit weights of the aggregates with moisture contents as used were determined from the weight of a $\frac{1}{4}$ cubic foot

Table 2. Sieve Analyses of Aggregates

Aggregate	Mix	Per cent by weight retained on Tyler sieves: No. 3/4″ sieve	No. 3/8″ sieve	No. 4 sieve	No. 8 sieve	No. 14 sieve	No. 28 sieve	No. 48 sieve	No. 100 sieve	Fineness modulus
Lite-Rock										
Coarse	A	....	54	99	100	....	....	....	....	6.53
Fine	A	....	....	14	32	46	61	85	98	3.36
Coarse	B	....	54	99	100	....	....	....	....	6.53
Fine	B	....	....	14	32	46	61	85	98	3.36
Combined	C	....	....	32	50	69	88	99	100	4.36
Combined	D	....	....	35	55	70	83	97	100	4.40
Combined	C_r	....	....	7	25	46	70	92	100	3.40
Combined	D_r	....	....	7	25	46	70	92	100	3.40
Coarse	E	....	1	88	96	98	99	99	100	5.81
Fine	E	....	....	14	33	56	81	97	99	3.80
Gravel										
Coarse	G	31	78	93	95	96	97	99	100	6.89
Fine	G	....	....	7	26	41	59	89	99	3.21
Expanded shale No. 2										
Coarse	H	....	....	84	99	99	99	99	100	5.80
Fine	H	....	....	....	29	56	76	87	93	3.41

Table 3. PHYSICAL PROPERTIES OF AGGREGATES

Aggregate	Mix	Unit wt, rodded, lb per cu ft	Moisture content, per cent by wt	Bulk specific gravity	24-hr absorption, per cent	
					By weight	By volume
Lite-Rock						
Coarse	A	30.6	0.3	......	......	....
Fine	A	49.9	6.6	......	......	....
Coarse	B	30.6	0.3	......	......	....
Fine	B	49.9	6.6	......	......	....
Combined..	C	44.2	2.0	......	......	....
Combined..	D	46.2	2.2	......	......	....
Combined..	C_r	48.6	0.0	......	......	....
Combined..	D_r	48.6	0.0	......	......	....
Coarse	E	30.9	0.0	0.80	13.4	6.7
Fine	E	43.0	2.0	1.14	14.9	10.3
Gravel						
Coarse	G	108.1	1.1	2.58	1.5	2.6
Fine	G	105.8	1.5	2.51	3.0	5.0
Expanded shale No. 2						
Coarse	H	44.3	0.0	1.31	5.7	4.1
Fine	H	74.0	0.1	1.82	7.5	9.0

measure of the aggregate rodded as described in ASTM Designation: C 29-42. The unit weights of the aggregates, along with other physical properties, are listed in Table 3. Lite-Rock weighs about two-thirds as much as the expanded shale No. 2.

5. **Specific gravity and absorption.** The determination of bulk specific gravity and twenty-four hour absorption for the aggregates was carried out as described in ASTM Designation: C 128-42 as far as possible. In other light-weight aggregate studies (1, p 5: 2, p 11) special, and in some cases elaborate, techniques have been found necessary for determination of specific gravity and absorption due to the difficulty in obtaining a saturated-surface-dry condition. However, the Lite-Rock was sufficiently like sand and gravel to preclude the need for special treatment which would have been required here only for the expanded shale No. 2. Since the investigation was principally concerned with the Lite-Rock, such painstaking methods were not thought justified.

Standard procedures were therefore followed with two exceptions: The Dunagan apparatus, which is supplied with a pail rather than the specified wire basket, was used to weigh the coarse aggre-

Figure 10. Coarse expanded shale No. 2 aggregate (actual size).

gate immersed. The fine light-weight aggregates were considered saturated-surface-dry when they would flow freely through the fingers though they would not respond to the slump test at this point.

The Lite-Rock aggregate, being coated throughout all sizes, approximated the slump condition when considered saturated-surface-dry, but the expanded shale No. 2 was quite harsh and was not suitable for the slump test.

Repeated determinations for bulk specific gravity showed agreement within 0.01 except for the expanded shale No. 2 for which the same technique gave agreement within 0.03. For the absorption test repeated determinations gave agreement within 0.2 per cent absorption except for the expanded shale No. 2 which gave values agreeing within 0.3 per cent for the coarse and 0.8 per cent for the fine aggregate. Mean values are reported in Table 3.

Figure 11. Fine expanded shale No. 2 aggregate (actual size).

IV. PROPORTIONING AND MIXING

A summary of mix data is given in Table 4. The data are tabulated completely in the Appendix.

1. **Maximum size.** Proportioning of Lite-Rock aggregate is complicated by the structural weakness of larger sizes. While it is desirable to avoid an oversanded mix as uneconomical, it is also necessary to limit the amount of coarse aggregate since compressive strength for light-weight aggregate concrete is a direct function of the aggregate strength.

With this in mind, $\frac{3}{4}$-inch aggregate was used in the two leaner mixes, A and B, while $\frac{3}{8}$-inch aggregate was used in the seven and nine sack mixes, C and D, as well as in the seven-sack mix, E. For further study of the effect of maximum aggregate size, seven-sack and nine-sack mixes, C_f and D_f, were made with a maximum aggregate size of $\frac{1}{4}$ inch.

Table 4. Mix Data

	Mix designation								
	Lite-Rock							Gravel	Expanded shale No. 2
	A	B	C	D	C_f	D_f	E	G	H
Cement factor	3.7	5.4	6.9	9.2	6.9	8.8	7.1	4.8	6.9
Maximum size aggregate, inches	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{3}{8}$	1	$\frac{3}{8}$
Per cent coarse, by weight	20	30	32	35	....	....	20	55	26
Dispersing agent	Yes	Yes	Yes	Yes	Yes	Yes	No	Yes	Yes
Water-cement ratio, by weight	1.07	0.68	0.49	0.40	0.64	0.47	0.61	0.61	0.55
Slump, in.	0.3	2.3	3.0	5.0	4.6	5.3	1.8	5.3	2.1
Fresh weight, lb cu ft	76.4	79.9	75.2	84.8	83.0	86.5	80.3	143.8	99.9

In the comparison mixes, the maximum size used was that considered most likely to occur in practice. The gravel was one-inch maximum and the expanded shale No. 2 was ⅜-inch as furnished from the plant.

2. **Proportions.** After deciding on the maximum size aggregate, further design was controlled by workability. In the two leanest mixes, as much coarse aggregate was used as compatible with workability, but in C, D, and E mixes the coarse was limited somewhat beyond the requirements for workability to gain greater aggregate strength. In mixes C_f and D_f, one grade of aggregate was used with no attempt to separate and recombine into an ideal gradation.

For the gravel mix, proportions were taken from the Portland Cement Association publication, "Design and Control of Concrete Mixtures" (3, p 18). These proportions were modified slightly after trial batches were made. Literature was also available for proportioning the expanded shale No. 2. Trial batches were made here also and a mix was used which contained a somewhat larger percentage of fines than suggested by the literature.

3. **Dry batching.** All aggregates were dry batched and were not soaked prior to mixing. This was contrary to the generally accepted view that light-weight aggregates should be saturated when used, or soaked for a time in the mixer. The principal reason for the soaking is to avoid drying out of the concrete batch due to absorption after discharge from the mixer. This practice had been followed in the preliminary tests and is no doubt necessary for highly absorptive aggregates but little difficulty was encountered here from drying out. Greater strength is claimed by one writer for moist aggregates, but an examination of his results shows this to be due to a higher cement factor obtained when bulking of the volume-measured moist aggregate resulted in a richer mix.

4. **Dispersing agent.** A commercial dispersing agent ("Pozzolith") was recommended by the manufacturer, and this was used for all the mixes except one. One-half pound of the dispersing agent per sack of cement was dissolved in a portion of the mixing water, and was very effective in producing a workable mix. An examination of Table 4 will show also that 25 per cent more water was required for the mix without the agent, than for a comparable mix where it was used.

5. **Mixing water.** The water-cement ratio law has been declared impracticable for mix design with light-weight aggregates because of high absorption and varying rate of absorption with dif-

ferent screen sizes (2, p 631). The water-cement ratio was recorded, however, and its effect will be discussed with the strength tests. The criterion used for water content was workability suitable for placing with mechanical vibration.

6. **Mixing.** Mixing was accomplished in a 1½-cubic foot tilt-drum mixer. While 1½-cubic foot batches of the gravel concrete could be mixed readily, the Lite-Rock aggregate was found to clog the mixer in this quantity, and was mixed in batches of 1 cubic foot or less. The comparison shale was also mixed in the smaller batches.

The light-weight aggregates require greater fall in the mixer for equal effectiveness in mixing. This was accomplished by lowering the drum to a more nearly horizontal position. Mixing time was about 5 minutes for all mixes except the two leanest, A and G, which were mixed 8 and 10 minutes respectively. This longer mixing time, which would not be necessary with the more thorough mixing obtained in a large mixer, served to bring out the action of the dispersing agent. The gravel mixture was quite dry until near the end of the mixing period.

7. **Workability and slump.** In general, satisfactory workability was obtained with a slump of about 4 inches. However, with the leanest mix, workability was obtained though there was practically no slump. In this mixture there was not sufficient cement paste to lubricate the surfaces, but the mix was wet enough to respond to vibration. Some tendency was shown toward drying out in the mixtures where all fine aggregate was used, and greater slump was required in these mixes to provide equally plastic concrete.

Two factors are present to alter the evaluation of slump with light-weight concrete: There is less weight to overcome cohesive forces and cause slump, and the significance of a slump test may be destroyed by subsequent drying out. Thus the slump test is not a complete description of consistency. In this work the consistencies obtained for Lite-Rock, gravel, and the expanded shale No. 2 mixes were very comparable.

8. **Vibration.** Lite-Rock concrete does not consolidate as readily as gravel concrete due to some harshness and lack of weight. This is also true of the comparison shale. Therefore, a small vibrator was used in the 6-inch cylinders as well as for the larger specimens. It was used in the same way in the measuring bucket which served to determine unit weights and cement factors.

For the 4-inch cylinders and other small specimens, a vibration table was improvised. The table was supported on rubber isolators, and vibrated by clamping to the table top the same vibrator used with the larger specimens.

9. **Measurements.** Most concrete materials were weighed on scales graduated to $\frac{1}{8}$ pound. Small quantities were weighed on balance scales graduated to 0.01 pound. The fresh concrete was placed in a volumetric measure calibrated at 0.2 cubic feet. This was weighed on the same scales used for the concrete materials. Cement factors were determined and are reported to the nearest 0.1 sack per cubic yard.

V. CONCRETE TESTS

1. **Curing.** The purpose of the testing program was to furnish data of practical value, and curing conditions were chosen accordingly. The specimens to be used for sonic and static modulus of elasticity tests were given a full 28-day moist-room cure at 70 F and 100 per cent relative humidity, as also was one set of 4 in. x 8 in. cylinders for comparison. All other specimens were given only a 7-day moist cure to correspond more closely with job practice. The remainder of the curing was accomplished in room air at about 70 F and 50 per cent relative humidity.

2. **Compressive strength tests (No. 1-5).** Compressive strength tests were made at 7, 28, and 90 days. Three tests were made at the 28-day age to furnish a comparison of curing condition effects, and a comparison between strength of 4-inch and 6-inch cylinders. Results of compressive strength tests are summarized in Table 5. Complete data are given in the Appendix.

At the end of the curing period, cylinders were weighed and dimensions were taken to the nearest 0.01 inch. Cylinders were then capped with leadite and tested in a 150,000-pound Riehle testing machine at a free-head-travel speed of 0.055 inch per minute. Moist cylinders were tested wet. The type of break was recorded and the amount of broken aggregate estimated. Compressive strength was determined to the nearest 10 lb/sq in.

3. **Compression tests (No. 4).** Tests for modulus of elasticity were made on all of the 6-inch cylinders. The apparatus used was a strainometer with a dial gage reading to 0.001 inch. This device, set up on a specimen at a 10-inch gage length, is shown in Figure 12. The testing was done on the 150,000-pound Riehle machine at a maximum speed of 0.055 inch per minute. The load was applied in 3,000-pound increments and strain readings were taken at each

Table 5. Results of Compressive Strength Tests

Test Series No. 1-5

Mix	Cement factor	Water-cement ratio, by weight	Slump, in.	Unit weight lb/cu ft		Compressive strength, lb/sq in					Ratio, strength-weight, psi/lb
						4″ x 8″ cylinders				6″ x 12″ cylinder	
				Fresh	Oven dry	7 day moist	7 day moist, 21 day dry	28 day moist	7 day moist, 83 day dry	28 day moist	
A	3.7	1.07	0.3	76.4	61.8	780	1,200	1,060	1,370	1,200	15.7
B	5.4	0.68	2.3	79.9	64.5	1,670	2,050	1,960	2,090	2,170	27.2
C	6.9	0.49	3.0	75.2	......	2,270	2,670	2,590	2,490	2,430	32.3
D	9.2	0.40	5.0	84.8	......	2,810	2,860	2,890	3,020	3,390	40.0
C_r	6.9	0.64	4.6	83.0	70.8	2,180	2,880	2,720	3,090	2,750	33.2
D_r	8.8	0.47	5.3	86.5	76.3	3,390	3,480	3,770	4,050	4,220	48.8
E	7.1	0.61	1.8	80.3	67.5	1,980	2,500	2,480	2,790	2,250	28.0
G	4.8	0.61	5.3	143.8	136.3	2,030	3,100	3,160	2,880	3,380	23.5
H	6.9	0.55	2.1	99.9	87.0	1,830	3,080	2,900	3,080	3,570	35.7

Note: Each test value is the average of three specimens.

increment. This was continued until approximately two-thirds of the ultimate load was reached. The strainometer was then removed and the specimen loaded until failure. Stress-strain curves are shown in Figure 13, and values for the secant modulus of elasticity, taken at 0.45 f'_c are plotted in Figure 14. Complete data for the compression tests are included in the Appendix.

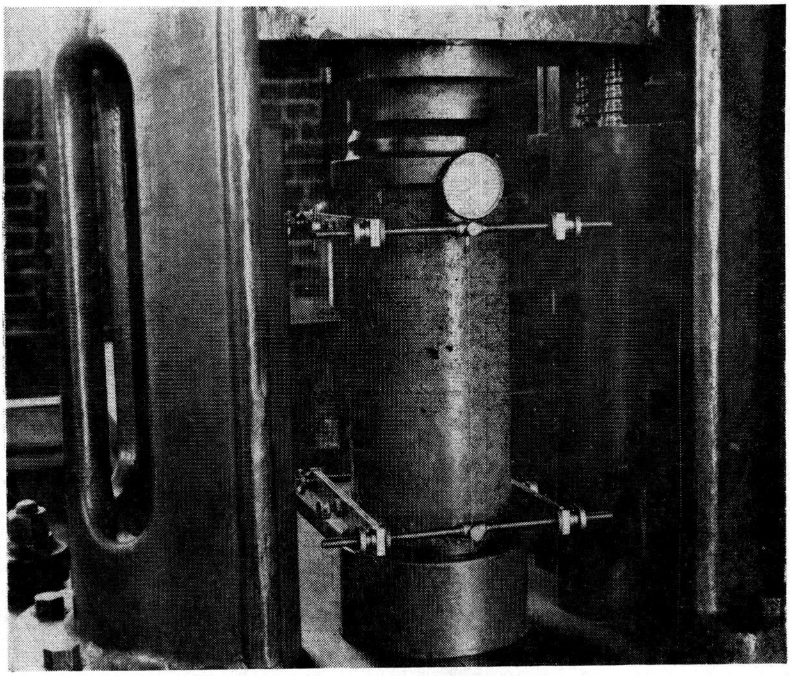

Figure 12. Compression test cylinder in testing machine with strainometer in place.

4. **Sonic modulus tests (No. 6_s).** The tests for flexure and for sonic modulus of elasticity were made on 6 in. x 6 in. x 36 in. plain-concrete beams cured moist. At 28 days the specimens were removed from the fog room, weighed, and placed on the sonic modulus tester.

This apparatus, which is shown in Figure 15, sets up a vibration by means of a variable-frequency audio oscillator. The oscillator furnishes an impulse which is transmitted to the beam by means of a driver placed at one end of the beam. The vibration thus set up is indicated in frequency and amplitude by a crystal pick-up placed at

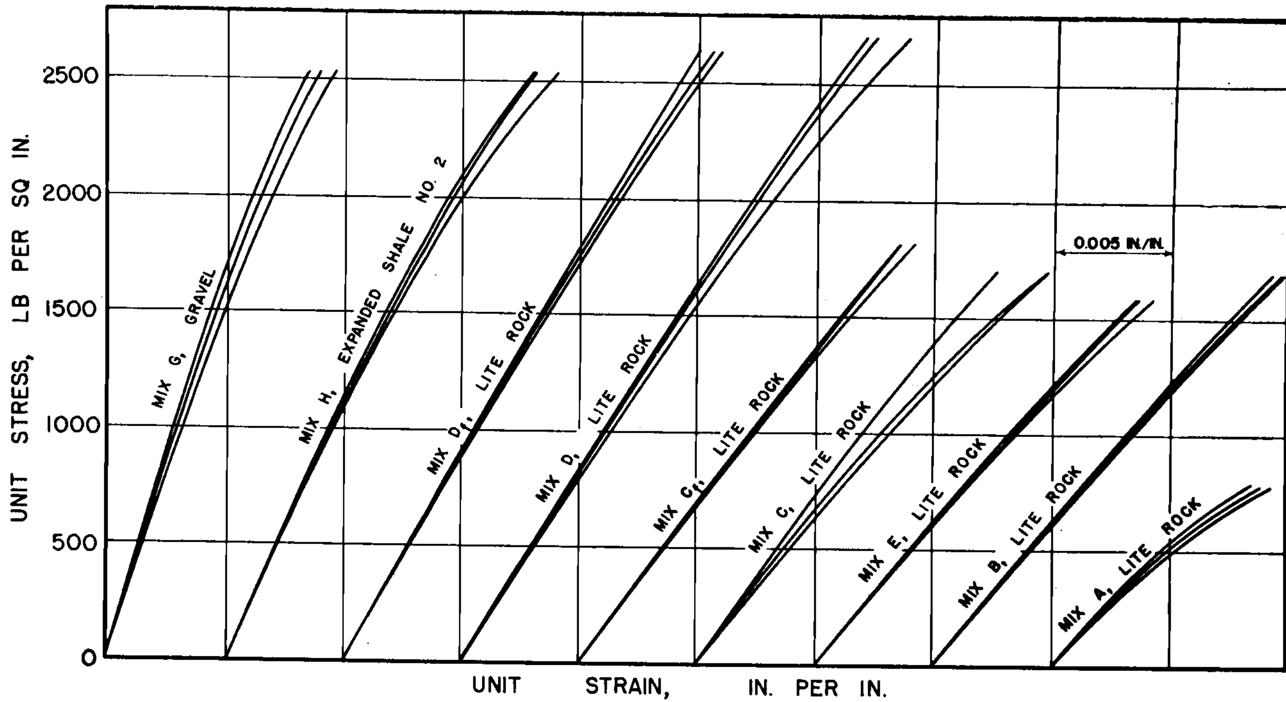

Figure 13. Stress-strain curves for all mixes tested. (Three cylinders of each.)

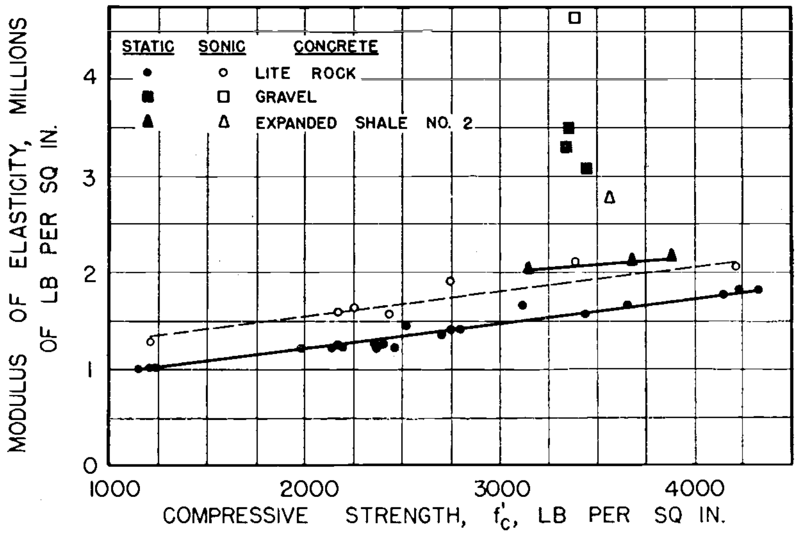

Figure 14. Static and sonic moduli of elasticity versus strength for three types of concrete.

the opposite end. The pick-up carries vibrations to the audio amplifier which then sends them to the oscilloscope where the vibration is indicated.

The lowest natural frequency is determined as the vibration which produces resonance and has nodal points only at the supports. The nodal points may be located by moving the pick-up along the beam and observing the points of minimum amplitude as indicated by the oscilloscope.

A dial reading from the apparatus corresponds to a certain frequency which is found from a calibration curve where it is plotted as a function of dial reading. The frequency is then inserted in the formula below to obtain the sonic modulus of elasticity, E_s.

$$E_s = \frac{Wl^3(1.2)f^2}{4.08bd^3},$$

Where W = weight of specimen in pounds,
l = length in inches,
b = width in inches,
d = depth in inches,
and f = frequency in cycles per second.

Results of the sonic modulus test are plotted in Figure 14 along with static modulus of elasticity.

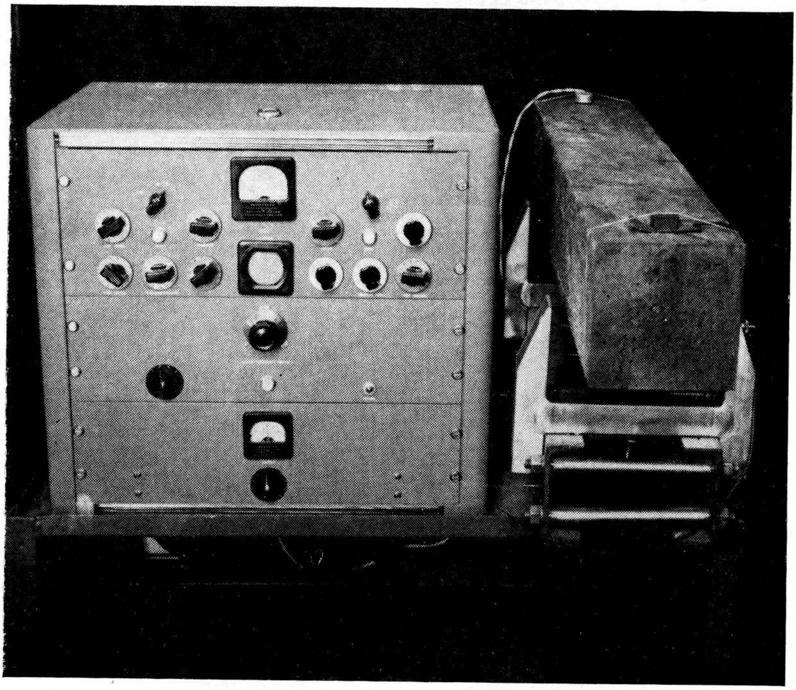

Figure 15. Sonic modulus test apparatus for plain concrete beams.

5. **Flexure tests (No. 6).** Immediately following the sonic modulus test, the specimen was removed and tested in flexure. A beam tester, made by the American Beam Testing Company, was used. This device provides third-point loading on an 18-inch span and a gage which reads modulus of rupture for a 6 in. x 6 in. beam directly in pounds per square inch. The apparatus is pictured in Figure 16. Two breaks were made on each 36-inch beam and the modulus of rupture was recorded to the nearest 10 pounds per square inch. Average values for modulus of rupture are given in Table 6. They are plotted in Figure 17. Complete data are tabulated in the Appendix.

6. **Bond tests (No. 7).** Specimens for bond pull-out tests were 8 in. x 8 in. cylinders with $\frac{5}{8}$-inch deformed bars extending about 20 inches below the bottom of the cylinder. The specimens were poured

Table 6. RESULTS OF FLEXURE TEST (NO. 6)

Modulus of rupture, lb per sq in								
Mix A	Mix B	Mix C	Mix D	Mix C_f	Mix D_f	Mix E	Mix G	Mix H
200	330	400	440	490	515	455	460	505

Each value is the average of two breaks.

on a bench with holes provided for the reinforcing steel. They were cured 7 days moist and 21 days in room air. To measure the initial end slip, a dial gage graduated to 0.0005 inch was used. A specimen ready for testing is shown in Figure 18.

A 50,000-pound Olsen testing machine was used for the pull-out tests with the lower portion of the load applied at 0.176 inch per

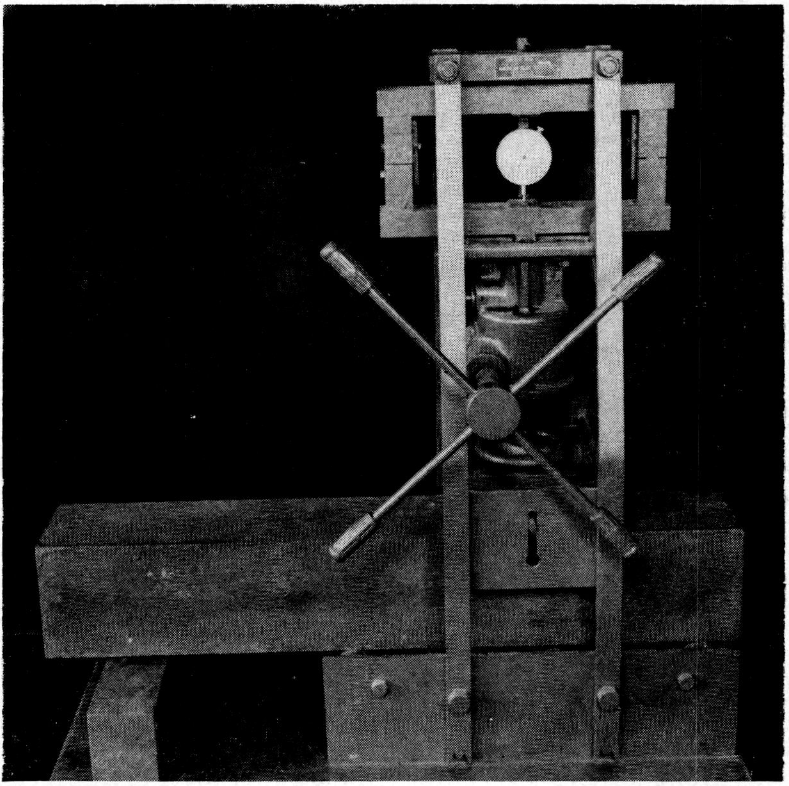

Figure 16. Flexure test on plain concrete beam.

minute. Loads were recorded at end slip of 0.001 inch, and at the ultimate value. Results from the pull-out tests are shown in Table 7, plotted in Figure 19, and given in detail in the Appendix.

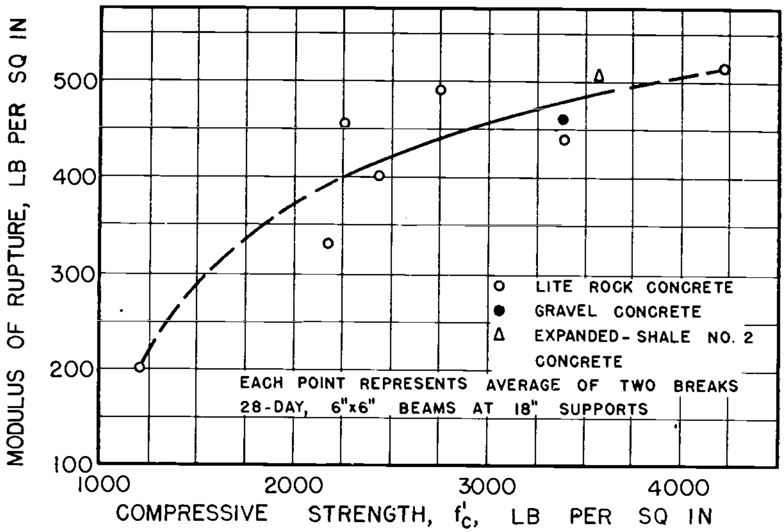

Figure 17. Relation between modulus of rupture and compressive strength.

Nearly all of the specimens failed due to splitting before the ultimate bond strength was reached. With the heavier concretes two of the gravel and one of the comparison shale specimens failed from tension in the steel. However, none of the specimens failed below the significant bond-stress at end slip.

7. **Dorry abrasion tests (No. 8).** Specimens for the abrasion test were 2 in. x 4 in. cylinders cured 7 days moist and 21 days in air. The abrasive material was crushed quartz between 30 and 40 mesh size. The abrasive was fed to a grinding disk which rotated approximately 30 times per minute. One thousand revolutions constituted a test.

The Dorry abrasion machine, which is shown in Figure 20, holds two specimens and it was originally intended to test two of each mix. However, the control on the flow of abrasive sand is not positive and results were not reliable. Therefore one specimen of each batch was tested opposite a gravel concrete specimen to provide a standard comparison.

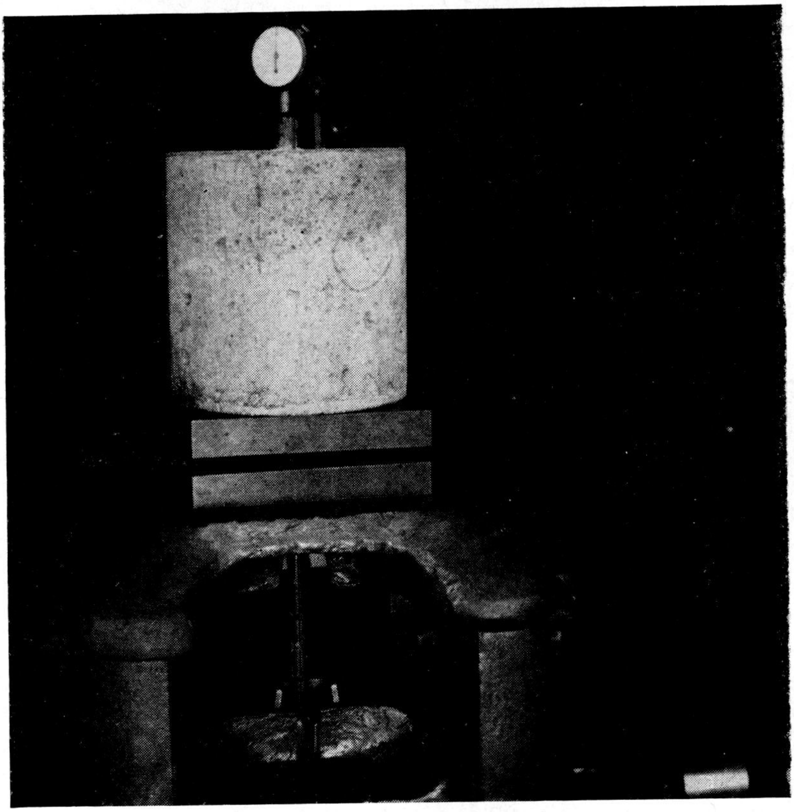

Figure 18. Bond test on pull-out specimen.

Table 7. RESULTS OF BOND TEST

	Average bond stress, lb/sq in.								
	Mix A	Mix B	Mix C	Mix D	Mix C_f	Mix D_f	Mix E	Mix G	Mix H
At end slip	256	351	528	523	426	572	392	317	549
At failure	532	605	729	777	700	842	633	1,163†	1,178*

Each value is the average for three specimens.
* Steel failed in one specimen.
† Steel failed in two specimens.

Roughness was ground off the specimens before testing and they were then subjected to 1,000 revolutions on the machine. They were next transferred to the opposite holder, turned end for end, and given a second 1,000 revolutions. The average loss in grams for 1,000 revolutions is recorded in Table 8.

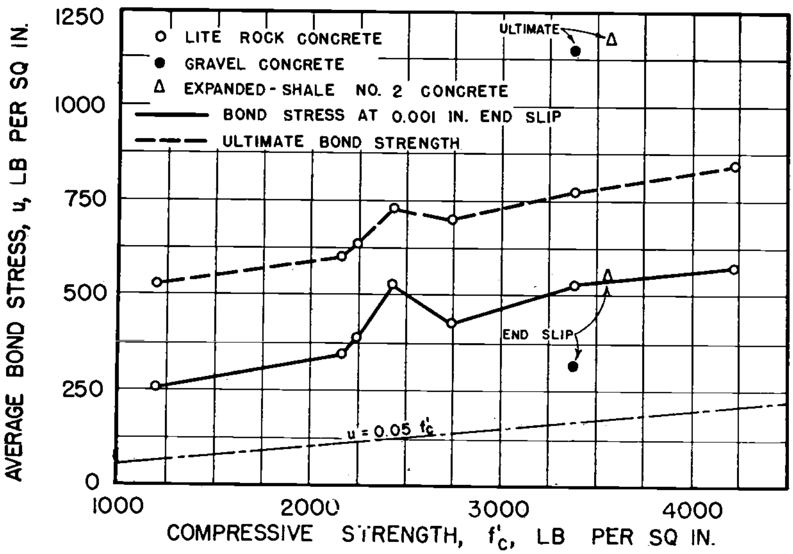

Figure 19. Relation between bond and compressive strength.

Table 8. RESULTS OF DORRY ABRASION TEST

Specimens: 2 in. x 4 in. cylinders — Curing: 7 days moist, 21 days air

Mix	Average weight loss in 1,000 revolutions of machine, grams	
	Tested specimen	Gravel comparison specimen
A	47.5	4.8
B	31.7	3.6
C_f	30.3	4.1
D_f	17.4	4.2
E	25.5	3.8
H	12.2	4.0

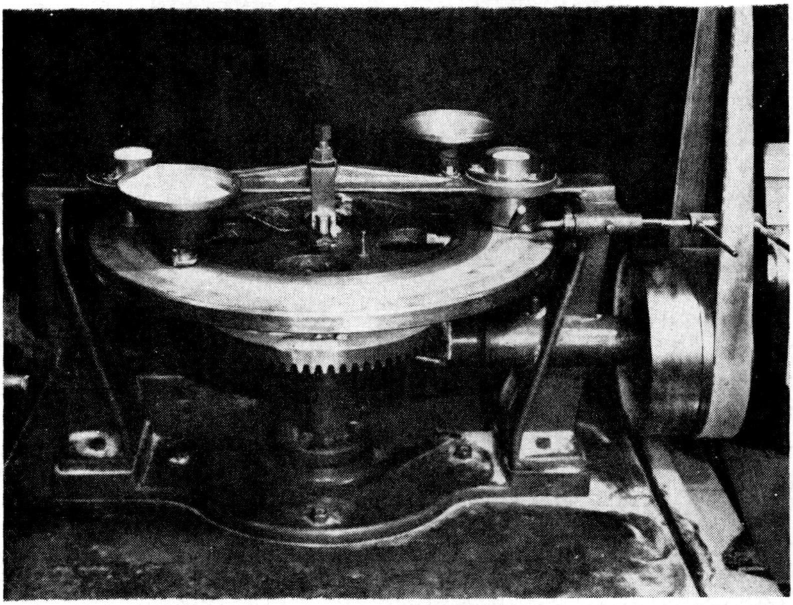

Figure 20. Dorry abrasion test on two-inch cylinders.

8. **Absorption tests (No. 9).** Specimens for the absorption test were 4 in. x 8 in. cylinders, cured moist for 7 days and in air for 21 days. At the close of the curing period, specimens were oven dried to constant weight, cooled, weighed, and immersed for 24 hours in water at 70 F. They were then removed from the water, wiped off with a cloth, and weighed. A summary of the absorption tests is shown in Table 9, and complete data are tabulated in the Appendix.

Table 9. SUMMARY OF ABSORPTION TEST RESULTS

Specimens: 4 in. x 8 in. cylinders — Curing: 7 days moist, 21 days air

	24-hour absorption, per cent						
	Mix A	Mix B	Mix C_f	Mix D_f	Mix E	Mix G	Mix H
By weight..	19.1	13.3	13.2	11.5	14.9	5.8	11.0
By volume..	18.7	13.8	14.9	14.1	16.1	12.8	15.3

Each value is the average for three specimens.

9. **Shrinkage tests (No. 10).** Specimens for the shrinkage test were 3 in. x 3 in. x 11 in. bars into which $\frac{1}{8}$-inch brass machine screws had been set for gage points at a 10-inch gage length. The brass screws had been drilled with a No. 60 drill for the strain gage which was used to measure shrinkage. The strain gage was graduated to 0.0001 inch and was checked against a standard 10-inch invar bar. Readings could be repeated on this bar within 0.0001 inch. A measurement is illustrated in Figure 21.

Shrinkage specimens were measured at 1 day and at 28 days. They were then oven dried, cooled, and measured again. Curing was 7 days moist, and 21 days in air. During curing the bars were placed on end where air could circulate about them freely.

Some of the gage-point screws showed instability as is reflected by the data tabulated in the Appendix. A summary of shrinkage test results is shown in Table 10.

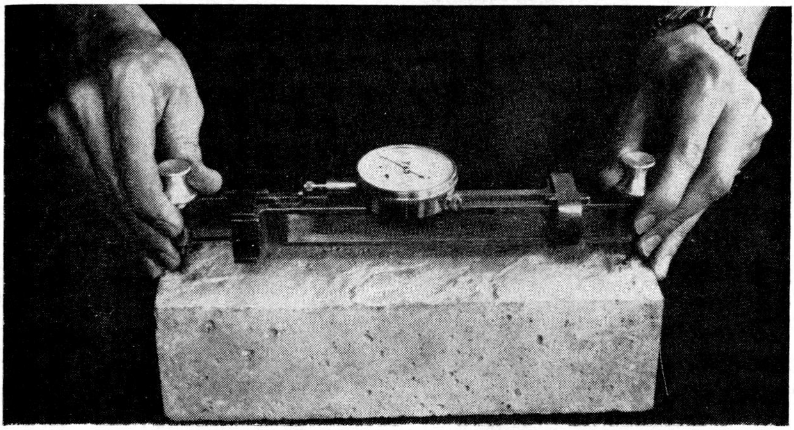

Figure 21. Measurement of shrinkage with strain gage.

Table 10. SUMMARY OF SHRINKAGE TEST RESULTS

Specimens: 3 in. x 3 in. x 11 in. bars — Curing: 7 days moist, 21 days air

Condition	Shrinkage, per cent						
	Mix A	Mix B	Mix C_f	Mix D_f	Mix E	Mix G	Mix H
28-day curing	0.027	0.029	0.036	0.027	0.029	0.046	0.057
Oven dry	0.047	0.053	0.068	*	0.061	0.094	*

Each value is the average for three specimens.
*Oven overheated with these specimens.

VI. DISCUSSION

1. **Mix design.** The design of a Lite-Rock concrete mixture differs from that for heavy concrete because of one principal factor, the inherent weakness of the larger aggregate. For this reason it is not safe to design a mixture on the basis of water-cement ratio. This is not to refute the application of the water-cement ratio law. This law does apply and, excluding mixes C_f and D_f because of their fineness and consequent higher absorption, a good curve could be drawn for 7-day compressive strength as a function of water-cement ratio. For the 28-day curing period, however, the comparatively weak aggregate cannot match the cement paste strength, and the water-cement ratio is of less significance than aggregate strength. It is therefore necessary to give consideration to the maximum size and the amount of coarse aggregate in regard to strength as well as to their effect on workability of the mixture.

From the results of these tests we may expect to produce 2,000-pound concrete with $\frac{3}{4}$-inch aggregate, about 40 per cent of which is retained on a No. 4 sieve; 3,000-pound concrete with $\frac{3}{8}$-inch aggregate, about 35 per cent of which is retained on a No. 4 sieve; and 4,000-pound concrete with $\frac{1}{4}$-inch aggregate. Cement factors for these mixes should be about $5\frac{1}{2}$ sk/yd for the first, and 9 sk/yd for the second and third, these factors obtaining when approximately a 3-inch slump is used.

Other factors which need to be considered are the relative weights of fine and coarse aggregate, the use of an air entraining or dispersing agent, and a slight drying out which may be expected when a very fine gradation is used.

The relative unit weights of fine and coarse aggregate need be considered when the aggregate is proportioned by weight. The fine aggregate unit weight is about $1\frac{1}{2}$ times that of the coarse. Thus a proportion of coarse aggregate amounting to 30 per cent by weight is nearly 40 per cent by volume.

The use of an air entraining or dispersing agent is not necessary for workability only, as a very workable mix was obtained in mix E where none was used. There was also an absence of segregation and of bleeding in this mix. However, the use of such an agent would seem advisable for the reduction in mixing water made possible, and the resulting increase in strength.

Drying out of the mix may be expected when a heavily sanded mixture is used. This is not excessive, however, and it is thought that an additional inch of slump is sufficient allowance for subsequent stiffening of the mix due to drying out.

2. **Unit weight.** Obviously the utility of light-weight concrete is limited by the degree of lightness. Light-weight concretes range from about 30 to 125 pounds per cubic foot. Each weight group may have its particular usefulness, but it is clear that we must not consider strength apart from weight.

Lite-Rock does not make the strongest expanded shale concrete. According to the tests herein reported, it does, however, make concrete stronger than any reported either by the Bureau of Reclamation or the National Bureau of Standards (1, p 10, 14) of equal weight.

In a report on the Bureau of Reclamation tests (4, p 597), the following statement was made concerning weight:

> The strength of light-weight concrete is dependent on the strength of the aggregate particles and the richness of the mix, but in general no amount of cement will produce concretes having strengths above 1,000 psi for concretes weighing less than 50 lb per cu ft or above 2,000 psi for concretes weighing less than 80 lb per cu ft, dry weight.

Lite-Rock concrete is shown to be an exception to the foregoing statement by Figure 22, where strengths of five Lite-Rock concrete mixes are plotted against oven-dry weight. The Bureau of Reclamation curve in Figure 22 cannot be compared with the Lite-Rock directly as it is based on a constant cement factor. It is of interest, however, to note that the Lite-Rock concrete with 3.7 sacks of cement per cubic yard is shown to advantage over the Bureau curve for 6-sack per cubic yard concrete.

3. **Effect of age on compressive strength.** Because of weakness of the aggregate, Lite-Rock concrete shows less gain beyond seven days than does heavy concrete. The heavier comparison shale showed an excellent increase in strength from 7 to 28 days. Beyond 28 days, however, the Lite-Rock concrete showed slight gains in all but one series, while the heavier two concretes made no increase in strength.

4. **Comparison of 4-inch and 6-inch cylinders.** Results from the 4-inch cylinders were not as consistent as desired. Flaws on the cylinder walls of 4-inch cylinders have much larger effect, and it is difficult to prevent eccentricity in loading. Results from the 6-inch cylinders averaged about 7 per cent higher than from the 4-inch with similar curing even though these 6-inch specimens were loaded by increments for the compression test. Results from these standard specimens are used where comparisons are made with other properties.

5. **Modulus of elasticity.** The modulus of elasticity of Lite-Rock concrete is about half that of gravel concrete. The heavier comparison shale had a modulus of elasticity about two-thirds that of gravel concrete. The curve for sonic modulus values (Figure 14), showed good agreement with that for static modulus values. The modulus of elasticity of Lite-Rock concrete may be stated very closely as follows:

$$E(\text{lb/sq in.}) = 750{,}000 + 250\, f'_c$$

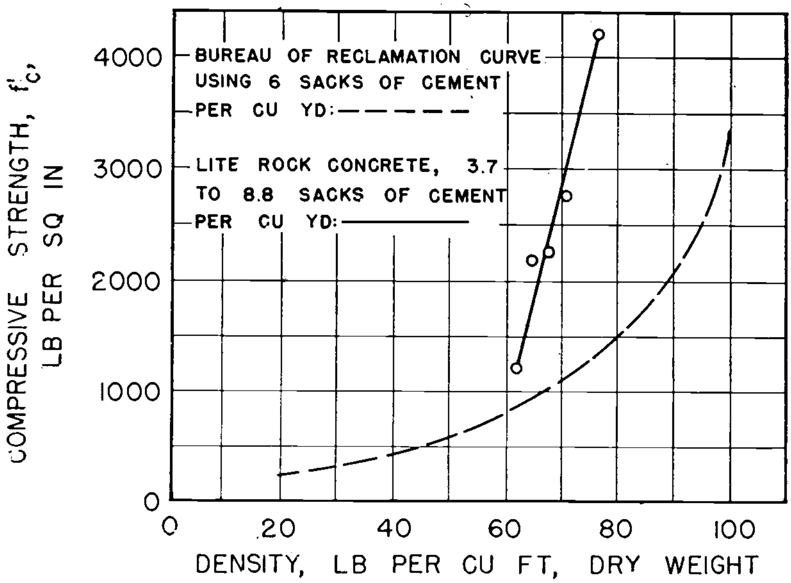

Figure 22. Strength of light-weight concrete as a function of density.

This value will be used in the part on Lite-Rock concrete design and the effect of the low modulus of elasticity will be brought out there.

6. **Flexural strength.** The flexural strength values of Lite-Rock concrete showed no distinct pattern but all were very good. The gravel concrete and the comparison shale concrete fell closely in line when they were plotted against compressive strength as in Figure 17.

7. **Bond strength.** Very satisfactory results were obtained from the bond pull-out tests as is shown in Figure 19. At initial

end slip of 0.001 in., both light-weight concretes showed about the same bond stress, and the gravel concrete was considerably lower. At failure, however, the heavier concretes went much higher than the Lite-Rock, and as has been noted, even caused steel failure in three cases. All results compare well with allowable values.

8. **Abrasion.** Lite-Rock concrete has little resistance to abrasion as shown by Table 8. The expanded-shale No. 2 concrete showed better resistance, but was still far under the gravel concrete. It should also be pointed out that this comparison is by weight and that a volume comparison would show the light-weight concretes even less satisfactory for abrasive resistance.

9. **Absorption.** A comparison of absorption based on dry weight is unfair to any light-weight concrete. A very light concrete may absorb 50 per cent of its own weight, while a heavy concrete could absorb the same amount of water and have only 10 per cent absorption by weight. Twenty-four-hour absorption values for Lite-Rock and the comparison shale concrete, shown in Table 9, were about the same, and were not greatly in excess of the gravel concrete when compared on a volume basis.

10. **Shrinkage.** The time allowed for shrinkage tests was insufficient to furnish final shrinkage values. However, the two comparison concretes furnish an index for the evaluation of shrinkage. The Lite-Rock concrete exhibited about two-thirds the shrinkage of gravel concrete both at 28 days, and when oven dry.

VII. DESIGN OF LITE-ROCK REINFORCED CONCRETE

NOTATION

b = width of rectangular beam or slab, inches.
d = depth from compression surface of beam or slab to center of tension steel, inches.
f_c = working stress in extreme fibers of concrete, psi.
f'_c = ultimate compressive stress, psi.
f_s = working stress in tension steel, psi.
I = moment of inertia of a section about the neutral axis, in.^{4}.
j = ratio of lever arm of resisting couple to depth, d.
k = ratio of depth of neutral axis to depth, d.
$K = \frac{1}{2} f_c kj = pf_s j$.
$n = \frac{E_s}{E_c}$ = ratio of modulus of elasticity of steel to that of concrete
$p = \frac{A_s}{bd}$ = ratio of tension steel area to effective area of concrete.
$r = \frac{f_s}{f_c}$ = ratio of stress in tension steel to compressive stress in extreme fiber of concrete.
u = average bond stress, psi.

The tests reported herein have discovered no weaknesses in Lite-Rock concrete with the exception of abrasive resistance. When a properly designed mix is used compressive strengths may be developed as desired; very adequate bond may be provided; and shear resistance, as shown by flexure tests, is in accord with compressive strengths. Shrinkage is low and absorption is not excessive. We are now to consider the adaptability of Lite-Rock concrete for use with reinforcing steel.

1. **Importance of weight in design.** The importance of the light weight of Lite-Rock concrete is readily appreciated. The light weight will be of major importance where the live load is equal to or less than the dead load. It will be of less importance where the live load is large in comparison to the dead load and the use of light-weight concrete may not always be justified in such cases.

2. **Effect of modulus of elasticity.** Another factor looms actually as large as the lightness in weight. This is the low modulus of elasticity. This will be apparent by comparison of Lite-Rock concrete design with that for gravel concrete $E = 1{,}000\, f'_c$ for the gravel and test values for the Lite-Rock. The two moduli are plotted in Figure 23. The value of $1{,}000\, f'_c$ was used in accord with conven-

tional design procedure but experimental values would serve equally well in bringing out the point of discussion.

In gravel concrete with balanced reinforcing the neutral axis falls at about three-eighths of the depth, *d*, below the surface of the

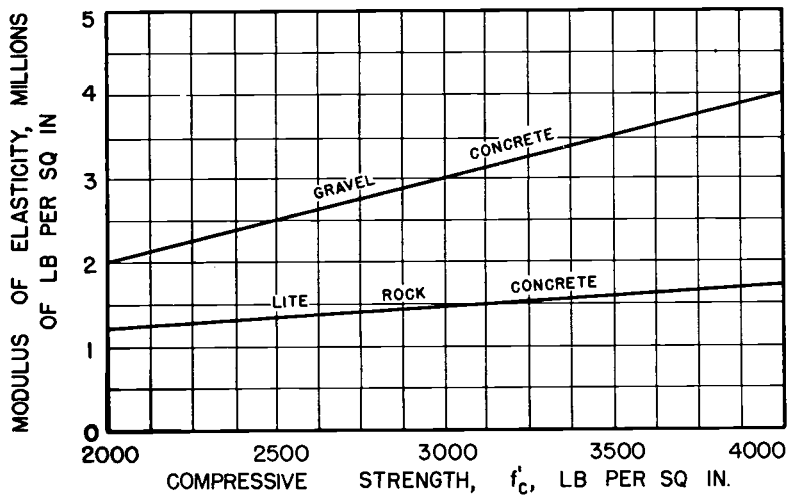

Figure 23. Relation of modulus of elasticity to strength.

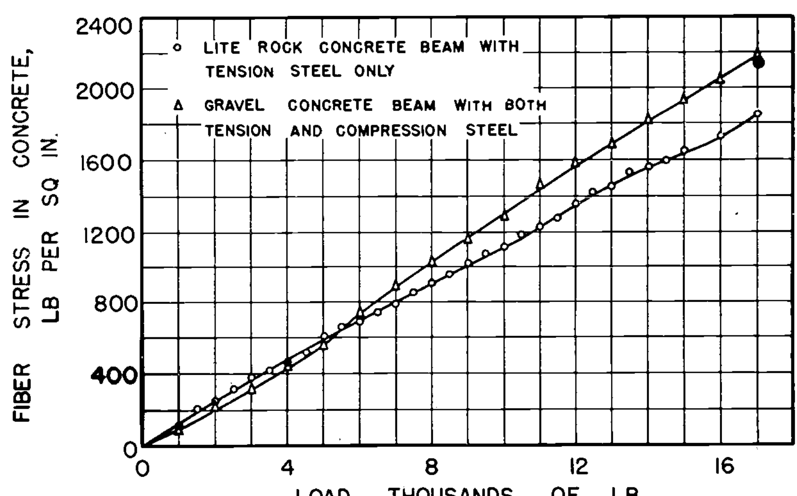

Figure 24. Moment factors for two types of concrete.

Table 11. DESIGN OF LITE-ROCK CONCRETE BEAMS AND SLABS

$k = \dfrac{n}{n+r}$ $\quad j = l - \dfrac{k}{3}$ $\quad p = \dfrac{n}{2r\,(n+r)}$ $\quad K = \tfrac{1}{2} f_c k j$ or $p f_s j$

(n) and f'_c	f_s	f_c	k	j	p	K
(18) 3,750	18,000	1,125	0.529	0.824	0.0165	245
		1,500	0.600	0.800	0.0250	360
		1,688	0.628	0.791	0.0295	420
	20,000	1,125	0.503	0.832	0.0141	235
		1,500	0.575	0.808	0.0216	349
		1,688	0.603	0.799	0.0254	406
(20) 3,000	18,000	900	0.500	0.833	0.0125	187
		1,200	0.571	0.810	0.0190	277
		1,350	0.600	0.800	0.0225	324
	20,000	900	0.474	0.842	0.0107	180
		1,200	0.545	0.818	0.0164	268
		1,350	0.575	0.808	0.0194	314
(22) 2,500	18,000	750	0.478	0.841	0.0100	151
		1,000	0.550	0.817	0.0153	225
		1,125	0.579	0.807	0.0181	263
	20,000	750	0.452	0.849	0.0085	144
		1,000	0.524	0.825	0.0131	216
		1,125	0.553	0.816	0.0156	254
(24) 2,000	18,000	600	0.444	0.852	0.0074	113
		800	0.516	0.828	0.0115	171
		900	0.545	0.818	0.0136	200
	20,000	600	0.419	0.860	0.0063	108
		800	0.490	0.837	0.0098	164
		900	0.519	0.827	0.0117	193

compression concrete. This means that only three-eighths of the concrete in the effective section is used to resist stress while the remainder is used merely to hold the steel in place.

With much larger n values for Lite-Rock concrete the neutral axis is shifted downward to about six-tenths of the depth below the surface of the compression concrete. Much more of the concrete becomes effective in compression and the neutral axis is placed midway (at $k = 0.6$) between the tension steel and the centroid of the compressive force. A higher percentage of steel is required for balanced reinforcing than with gravel concrete.

The value of this low modulus of elasticity is shown in Figure 24 where moment factors for the two types of concrete are compared.

Table 12. REVIEW OF LITE-ROCK CONCRETE BEAMS AND SLABS

$k = \sqrt{2pn + (pn)^2} - pn$ $j = l - \frac{1}{3}k$

p	$n = 18$		$n = 20$		$n = 22$		$n = 24$	
	k	j	k	j	k	j	k	j
0.001	0.173	0.942	0.181	0.941	0.189	0.937	0.196	0.935
0.002	0.235	0.922	0.246	0.918	0.256	0.915	0.266	0.911
0.003	0.279	0.907	0.292	0.913	0.303	0.899	0.314	0.895
0.004	0.314	0.895	0.328	0.891	0.341	0.886	0.353	0.882
0.005	0.344	0.885	0.358	0.881	0.372	0.876	0.384	0.872
0.006	0.369	0.877	0.384	0.872	0.398	0.867	0.412	0.863
0.007	0.392	0.869	0.407	0.864	0.422	0.859	0.435	0.855
0.008	0.412	0.863	0.428	0.857	0.443	0.852	0.457	0.848
0.009	0.430	0.857	0.446	0.851	0.462	0.846	0.476	0.841
0.010	0.446	0.851	0.463	0.846	0.479	0.840	0.493	0.836
0.011	0.462	0.846	0.479	0.840	0.495	0.835	0.509	0.830
0.012	0.476	0.841	0.493	0.836	0.509	0.830	0.524	0.825
0.013	0.489	0.837	0.507	0.831	0.523	0.826	0.537	0.821
0.014	0.501	0.833	0.519	0.827	0.535	0.822	0.550	0.817
0.015	0.513	0.829	0.531	0.823	0.547	0.818	0.562	0.813
0.016	0.524	0.825	0.542	0.819	0.558	0.814	0.573	0.809
0.017	0.534	0.822	0.552	0.816	0.568	0.811	0.583	0.806
0.018	0.544	0.819	0.562	0.813	0.578	0.807	0.593	0.802
0.019	0.553	0.816	0.571	0.810	0.587	0.804	0.602	0.799
0.020	0.562	0.813	0.580	0.807	0.596	0.801	0.611	0.796
0.021	0.570	0.810	0.588	0.804	0.605	0.798	0.619	0.794
0.022	0.578	0.807	0.596	0.801	0.612	0.796	0.627	0.791
0.023	0.586	0.805	0.604	0.799	0.620	0.793	0.635	0.788
0.024	0.593	0.802	0.611	0.796	0.627	0.791	0.642	0.786
0.025	0.600	0.800	0.618	0.794	0.634	0.789	0.649	0.784
0.026	0.607	0.798	0.625	0.792	0.641	0.786	0.656	0.781
0.027	0.613	0.796	0.631	0.790	0.647	0.784	0.662	0.779
0.028	0.619	0.794	0.637	0.788	0.653	0.782	0.668	0.777
0.029	0.625	0.792	0.643	0.786	0.659	0.789	0.674	0.775
0.030	0.631	0.790	0.649	0.784	0.665	0.778	0.679	0.774

From 25 to 35 per cent more moment is carried by the Lite-Rock concrete than by the gravel concrete of equal compressive strength.

3. **Design tables.** Factors for the design of rectangular beams and slabs with Lite-Rock concrete are given in Table 11. Factors for the review of beams are offered in Table 12.

4. **Senior beam tests.** It was necessary to discard some of the deformeter data on the beams poured by the senior students as it was not compatible. Therefore, the following comparison is limited to two beams using only the data which were considered reliable. How-

ever, the results available from tests made in the senior course in previous years are in agreement with the principle involved here.

The beam made of gravel concrete used in the comparison was reinforced both in tension and in compression. It had tension steel equal to the Lite-Rock beam and in addition two ⅞-inch round bars for compression reinforcement. The stresses in the concretes are plotted against load in Figure 25. The value of the low modulus of elasticity with the consequently greater *k* value is illustrated here to a conclusive degree.

5. **Deflection.** The question of deflection arises immediately when low modulus of elasticity, *E*, is mentioned. Greater deflection is expected with the lower modulus. In a homogeneous beam, deflection would increase as the value for *E* decreased.

This might lead us to expect a doubly large deflection for Lite-Rock concrete members. However, an investigation at the University

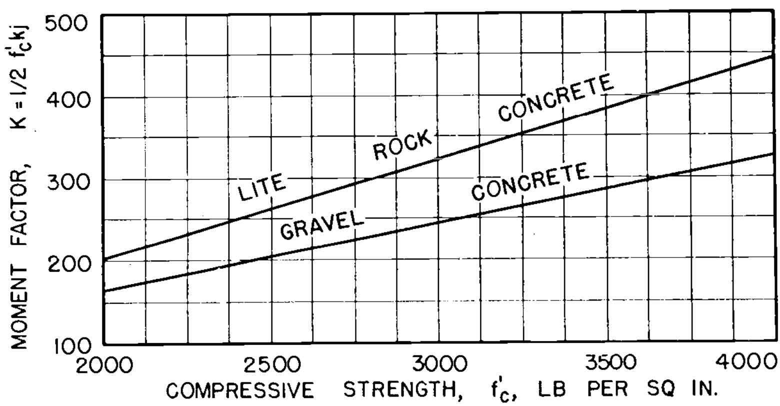

Figure 25. Comparison between reinforced concrete beams of Lite-Rock and gravel concretes.

of Illinois (5, p 76) showed only about 30 per cent more deflection for expanded shale beams than for gravel beams. In the beam tests conducted at Oregon State College by senior students more steel was used in Lite-Rock beams than in gravel beams in proportion to the requirements for balanced reinforcing. Here the Lite-Rock beams averaged 5 per cent more deflection than the gravel beams at a given load in the working range and 14 per cent less deflection at a given load near the ultimate. Equal reinforcement might be expected to agree more closely with the University of Illinois results.

This unexpected stiffness for expanded shale concrete must be explained as the result of an increased moment of inertia with the decreased modulus of elasticity, since deflection is controlled by the product of I and E. The value of I for a reinforced concrete member is not agreed upon in the literature. Some expressions for moment of inertia would give support to the experimental findings (5, p 76) while others would make the moment practically the same as if gravel concrete were used. The difference is in the consideration given to the concrete below the neutral axis. If this concrete is neglected the moment of inertia of a Lite-Rock member is much larger than that for one of gravel; if this tension concrete is figured the two I values are about equal. The writer would point out that the low modulus of elasticity of Lite-Rock concrete results in less cracking below the neutral axis since tension stresses would be only half the values for gravel concrete. Thus the smaller section area below the neutral axis in a Lite-Rock beam is probably as effective in deflection resistance as the larger section area in a gravel beam. Since the area above the neutral axis is considerably larger for a Lite-Rock beam, this would result in a larger moment of inertia, and account for the low deflections as observed.

6. **Increase in steel.** An increase in steel is required for balanced reinforcing with Lite-Rock concrete and this may raise a question as to economy. The steel requirement varies in a particular member with the value of j. The value of j decreases as k increases but only to the extent of one-third of the increase. Thus the loss of effectiveness of the steel is only slight as compared to the gain in effectiveness of the concrete.

VIII. CONCLUSIONS

The following conclusions may be drawn concerning Lite-Rock concrete:

1. Unit weight, dry, is from 60 to 80 pounds per cubic foot.
2. The maximum size and amount of coarse aggregate are critical in mix design.
3. An air-entraining agent or dispersing agent is recommended but not necessary.
4. The compressive strength ranges from 1,200 to 4,200 pounds per square inch depending on the cement factor and the maximum size aggregate.
5. Less strength is gained beyond the 7-day curing period than with heavier concrete.
6. Resistance to bond and shear is in accord with compressive strength.
7. Absorption is not excessive when considered on a volume basis.
8. Twenty-eight-day shrinkage is less than that for gravel concrete.
9. Abrasive resistance is very low.
10. The low modulus of elasticity of this concrete is remarkably well suited to reinforced concrete design.

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X. APPENDIX

The following tables present the test data in detail for the benefit of those who wish to make a more complete analysis of the results and conclusions.

Mix Data

	Mix A	Mix B	Mix C	Mix D	Mix C_r	Mix D_r	Mix E	Mix G	Mix H
Date poured	3–25–50	3–23–50	3–18–50	3–15–50	5–20–50	5–13–50	3–28–50	3–22–50	3–21–50
Proportions:									
Cement, lb	11.79	17.25	25.00	36.84	22.30	30.00	25.40	25.20	23.40
Fine aggregate, lb	36.00	31.50	32.90	43.50	40.10	40.10	33.00	78.60	44.40
Coarse aggregate, lb	9.00	13.50	8.40	2.72	....	....	8.25	97.20	15.96
Dispersing agent, lb	0.06	0.10	0.13	0.16	0.13	0.16	....	0.13	0.12
Water, lb	12.62	11.70	12.13	14.86	14.20	14.00	15.40	15.28	12.76
Total batch weight, lb	69.47	74.05	78.56	98.08	76.73	84.26	82.05	216.40	96.64
Approximate mixing time	8 min	5 min	5 min	5 min	5 min	5 min	5 min	10 min	5 min
Average slump, in.	0.3	2.3	3.0	5.0	4.6	5.3	1.8	5.3	2.1
Workability	Good	Very good	Very good	Very good	Very good	Very good	Very good	Very good	Very good
Bleeding	Yes	No	No	No	No	No	No	No	No
Segregation	No	No	No	No	No	No	No	No	No
Fresh wt, 0.2 cu ft	15.29	15.97	15.03	16.80	16.59	17.30	16.06	28.75	19.98
Unit wt, lb/cu ft	76.45	79.85	75.15	84.78	82.95	86.50	80.30	143.75	99.90
Cement factor, sk/cu yd	3.7	5.4	6.9	9.2	6.9	8.8	7.1	4.8	6.9
Moisture content, per cent dry wt									
Fine aggregate	6.6	6.6	2.4	2.2*	0.0	0.0	2.0	1.5	0.1
Coarse aggregate	0.3	0.3	0.2	....	0.0	0.0	0.0	1.1	0.0
Water-cement ratio by wt	1.07	0.68	0.49	0.40	0.64	0.47	0.61	0.61	0.55

* Combined.

Data on Seven-Day Compressive Strength Test

Test Series No. 1

Specimen: 4″ x 8″ cylinders

Curing: 7 days moist

Mix and specimen number	Date tested	Dimensions, in.		Weight, lb	Ultimate load, lb	Type of break	Per cent broken aggregate	f'_c, lb per sq in.
		Diameter	Height					
Mix A								
1	4- 1-50	4.00	8.04	4.37	10,690	Diagonal	25	850
2	4- 1-50	4.00	8.00	4.37	9,360	Cone	10	750
3	4- 1-50	4.00	7.92	4.27	9,360	Cone	10	750
Mix B								
1	3-30-50	3.98	8.12	4.65	19,200	Cone	60	1,540
2	3-30-50	2.97	7.92	4.56	20,190	Diagonal	70	1,630
3	3-30-50	4.00	8.05	4.85	23,240	Diagonal	60	1,850
Mix C								
1	3-25-50	3.96	8.10	4.54	26,870	Cone	75	2,180
2	3-25-50	3.97	8.06	4.52	29,020	Diagonal	75	2,340
3	3-25-50	4.03	8.02	4.56	29,410	Cone	75	2,300
Mix D								
1	3-24-50	3.99	8.15	5.82	34,760	Diagonal	75	2,780
2	3-24-50	4.00	8.10	4.74	34,090	Diagonal	75	2,710
3	3-24-50	4.00	8.14	4.82	37,080	Diagonal	75	2,950
Mix C_f								
1	5-27-50	3.98	8.08	4.77	27,350	Diagonal	50	2,200
2	5-27-50	4.00	8.06	4.78	25,990	Diagonal	50	2,070
3	5-27-50	3.97	8.06	4.78	27,940	Cone	50	2,260
Mix D_f								
1	5-20-50	4.01	8.06	5.03	43,050	....	....	3,410
2	5-20-50	3.99	8.16	5.02	44,170	....	....	3,530
3	5-20-50	3.99	8.08	4.93	40,240	....	....	3,220
Mix E								
1	4- 4-50	3.98	8.06	4.73	26,140	Diagonal	60	2,100
2	4- 4-50	3.97	8.05	4.53	22,280	Cone	60	1,800
3	4- 4-50	4.01	8.05	4.72	25,840	Diagonal	40	2,050
Mix G								
1	3-29-50	3.98	8.08	8.24	25,670	Cone	....	2,060
2	3-29-50	4.00	8.04	8.23	25,700	Cone	....	2,040
3	3-29-50	4.01	8.02	8.30	25,010	Cone	....	1,980
Mix H								
1	3-28-50	4.00	8.08	5.83	21,160	Cone	25	1,680
2	3-28-50	3.98	8.02	5.75	23,030	Diagonal	20	1,850
3	3-28-50	3.99	7.94	5.76	24,340	Diagonal	20	1,950

Data on Twenty-Eight Day Compressive Strength Test

Test Series No. 2

Specimen: 4″ x 8″ cylinders

Curing: 7 days moist, 21 days air

Mix and specimen number	Date tested	Dimensions, in.		28 day weight, lb	Ultimate load, lb	Type of break	Per cent broken aggregate	f'_c, lb per sq in.
		Diameter	Height					
Mix A								
1	4–22–50	4.00	8.04	3.94	13,450	Cone	10	1,070
2	4–22–50	4.06	8.06	4.11	16,910	Diagonal	50	1,310
3	4–22–50	3.96	8.06	3.98	14,980	Diagonal	40	1,220
Mix B								
1	4–20–50	3.98	8.00	4.19	25,490	Diagonal	70	2,050
2	4–20–50	3.98	8.04	4.33	24,990	Diagonal	70	2,010
3	4–20–50	3.96	8.02	4.23	25,860	Cone	70	2,100
Mix C								
1	4–15–50	3.98	8.04	4.28	32,570	Diagonal	70	2,620
2	4–15–50	3.97	8.06	4.31	33,400	Diagonal	70	2,700
3	4–15–50	3.97	8.05	4.34	33,480	Cone	70	2,700
Mix D								
1	4–14–50	3.99	8.05	4.61	36,790	Cone	90	2,940
2	4–14–50	3.99	7.94	4.56	36,990	Diagonal	90	2,960
3	4–14–50	3.99	8.08	4.62	33,650	Diagonal	90	2,690
Mix C_f								
1	6–17–50	4.00	8.00	4.45	36,660	Diagonal	50	2,920
2	6–17–50	4.02	8.04	4.45	35,900	Diagonal	50	2,830
3	6–17–50	3.99	8.02	4.33	36,070	Cone	50	2,890
Mix D_f								
1	6–10–50	4.02	8.06	4.81	34,420	Diagonal	50	2,710
2	6–10–50	4.05	8.02	4.95	47,230	Diagonal	50	3,670
3	6–10–50	4.00	8.10	4.87	51,180	Diagonal	50	4,070
Mix E								
1	4–25–50	3.96	8.04	4.59	33,370	Cone	70	2,710
2	4–25–50	4.02	8.08	4.54	30,070	Cone	70	2,370
3	4–25–50	3.97	8.07	4.49	29,970	Diagonal	50	2,420
Mix G								
1	4–19–50	3.98	7.96	7.71	38,050	Cone	....	3,060
2	4–19–50	4.05	8.06	8.16	40,800	Cone	....	3,170
3	4–19–50	3.97	8.02	7.79	37,920	Diagonal	....	3,060
Mix H								
1	4–18–50	4.00	8.03	5.59	38,220	Diagonal	40	3,040
2	4–18–50	3.98	8.04	5.72	38,790	Diagonal	40	3,120
3	4–18–50	3.97	8.10	5.66	38,070	Diagonal	40	3,080

Data on Twenty-Eight Day Compressive Strength Test

Test Series No. 3

Specimen: 4″ x 8″ cylinders — Curing: 28 days moist

Mix and specimen number	Date tested	Dimensions, in. Diameter	Dimensions, in. Height	28 day weight, lb	Ultimate load, lb	Type of break	Per cent broken aggregate	f'_c, lb per sq in.
Mix A								
1	4–22–50	4.00	8.04	4.44	11,420	Diagonal	15	910
2	4–22–50	3.94	8.06	4.37	13,480	Diagonal	25	1,110
3	4–22–50	4.00	8.06	4.47	14,660	Diagonal	35	1,170
Mix B								
1	4–20–50	3.98	8.02	4.80	24,630	Diagonal	70	1,990
2	4–20–50	3.97	7.92	4.65	24,130	Cone	70	1,950
3	4–20–50	3.97	8.12	4.71	24,130	Diagonal	70	1,950
Mix C								
1	4–15–50	3.98	8.10	4.53	31,990	Diagonal	50	2,570
2	4–15–50	3.98	8.12	4.54	32,030	Diagonal	70	2,570
3	4–15–50	3.94	8.06	4.49	32,020	Diagonal	60	2,630
Mix D								
1	4–14–50	4.00	8.08	4.90	37,840	Diagonal	90	3,010
2	4–14–50	3.97	8.08	4.84	33,060	Diagonal	75	2,670
3	4–14–50	3.97	8.08	4.88	37,040	Diagonal	90	2,990
Mix C_f								
1	6–17–50	4.01	8.12	4.77	32,270	Cone	50	2,560
2	6–17–50	3.97	8.02	4.64	33,840	Diagonal	50	2,730
3	6–17–50	4.00	8.06	4.74	36,100	Diagonal	50	2,870
Mix D_f								
1	6–10–50	3.97	8.04	4.92	48,210	Cone	50	3,890
2	6–10–50	3.98	8.02	4.85	48,960	Cone	50	3,940
3	6–10–50	3.98	8.02	4.92	43,450	Cone	50	3,490
Mix E								
1	4–25–50	3.97	8.02	4.86	33,720	Cone	70	2,720
2	4–25–50	4.00	8.06	4.72	29,880	Cone	70	2,380
3	4–25–50	3.97	8.02	4.67	29,000	Diagonal	50	2,340
Mix G								
1	4–19–50	3.99	8.06	8.26	40,220	Cone	....	3,220
2	4–19–50	4.01	8.10	8.25	39,810	Cone	....	3,150
3	4–19–50	3.98	8.08	8.28	38,580	Diagonal	....	3,100
Mix H								
1	4–18–50	4.00	8.08	5.91	34,860	Diagonal	40	2,770
2	4–18–50	3.98	8.00	5.87	37,660	Diagonal	40	3,030
3	4–18–50	3.99	8.00	5.86	36,170	Diagonal	40	2,890

RESULTS OF TEST SERIES No. 4

COMPRESSION

General data:

Specimens: 6″ x 12″ cylinders, moist cured 28 days, tested wet, gage length 10 inches.

Loading: Increments of 2,000 or 3,000 lb at maximum speed of 0.055 in. per minute.

Typical calculations:

Load, lb	$\frac{\text{Load}}{\text{Area}}$ Unit stress lb/sq in.	Gage reading 0.001 in.	Corrected gage reading 0.001 in.	$\frac{\text{Gage reading}}{2}$ Deformation 0.001 in.	$\frac{\text{Deformation}}{10}$ Unit strain 0.001 in./in.
2,000	71	1.0	1.2	0.6	0.06
4,000	143	2.2	2.4	1.2	0.12
6,000	214	3.5	3.7	1.95	0.195
8,000	285	4.8	5.0	2.5	0.25
10,000	356	6.1	6.3	3.15	0.315
12,000	427	7.6	7.8	3.9	0.39
14,000	498	9.2	9.4	4.7	0.47
16,000	568	10.6	10.8	5.4	0.54
18,000	639	12.5	12.7	6.35	0.635
20,000	710	14.5	14.7	7.35	0.735
22,000	781	16.4	16.6	8.3	0.83
34,110	$1{,}210 = f'_c$	Failure			

0.45 (ultimate load) = 15,580 lb $\quad$ $0.45\, f'_c = 545$

Unit strain at 0.45 (ultimate load) = 0.00053 in. per in.

Modulus of elasticity, E:

$$E = \frac{\text{Stress}}{\text{Strain}} = \frac{545}{53 \times 10^{-5}} = 1.03 \times 10^6 \text{ lb/in.}^2$$

Data on Compression Test

Test Series No. A-4

Date: 4-22-50

Load, lb	Cylinder No. 1 5.99″ x 11.94″		Cylinder No. 2 5.99″ x 12.02″		Cylinder No. 3 5.98″ x 12.00″	
	Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.	Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.	Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.
2,000	71	6.0	71	6.0	71	7.0
4,000	143	12.0	143	13.5	142	13.0
6,000	214	19.5	214	21.0	214	19.0
8,000	285	25.0	285	28.0	285	26.5
10,000	356	31.5	356	35.5	356	33.0
12,000	427	39.0	427	44.0	427	40.0
14,000	498	47.0	498	52.0	498	48.0
16,000	568	54.0	568	61.5	570	57.0
18,000	639	63.5	639	71.5	641	66.0
20,000	710	73.5	710	81.0	712	76.5
22,000	781	83.0	781	91.5	783	87.5
32,230	......	......	......	......	1,147	Failure
34,110	1,201	Failure	......	......	......	......
34,620	......	......	1,229	Failure	......	......
0.45 max	545	53.0	553	54.0	516	51.5
E, lb/in.^{2}	1.03×10^6		1.02×10^6		1.01×10^6	

Data on Compression Test

Test Series No. B-4

Date: 4-20-50

	Cylinder No. 1 5.98″ x 12.00″		Cylinder No. 2 5.97″ x 12.10″		Cylinder No. 3 5.98″ x 12.08″	
Load, lb	Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.	Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.	Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.
2,000	71	6.0	71	5.0	71	5.5
4,000	142	12.0	143	12.0	142	12.0
6,000	214	18.0	214	18.0	214	17.5
8,000	285	24.0	286	23.5	285	23.0
10,000	356	29.0	357	29.5	356	28.5
12,000	427	34.0	429	35.5	427	34.0
14,000	498	40.0	500	41.0	498	40.0
16,000	570	46.0	572	47.0	570	45.5
18,000	641	51.5	643	52.5	641	51.0
20,000	712	57.0	715	59.5	712	57.0
22,000	783	63.0	786	65.0	783	63.0
24,000	854	68.5	857	71.0	854	70.0
26,000	926	74.5	929	77.0	926	75.5
28,000	997	81.0	1,000	82.5	997	82.0
30,000	1,068	86.0	1,072	88.5	1,068	87.5
32,000	1,139	92.0	1,143	95.0	1,139	94.0
34,000	1,210	97.5	1,215	101.5	1,210	100.5
36,000	1,282	104.0	1,286	107.0	1,282	107.0
38,000	1,353	110.5	1,358	114.0	1,353	114.0
40,000	1,424	117.5	1,429	120.0	1,424	120.5
42,000	1,495	123.0	1,500	127.0	1,495	128.0
44,000	1,566	130.0	1,572	134.0	1,566	134.5
46,000	1,638	136.5	1,643	140.0	1,638	142.0
48,000	1,709	143.0	1,715	149.0	1,709	150.5
59,840	....	....	2,138	Failure	....	....
60,950	2,170	Failure	....	....	....	....
61,590	....	....	....	....	2,193	Failure
0.45 max	977	77.5	962	79.0	987	....
E, lb/in.^{2}	1.26×10^6		1.22×10^6		1.23×10^6	

Data on Compression Test

Test Series No. C-4

Date: 4-15-50

Load, lb	Cylinder No. 1 5.99″ x 12.02″ Unit stress, lb per sq in.	Cylinder No. 1 Unit strain, 10^{-5} in. per in.	Cylinder No. 2 5.98″ x 12.04″ Unit stress, lb per sq in.	Cylinder No. 2 Unit strain, 10^{-5} in. per in.	Cylinder No. 3 5.99″ x 12.02″ Unit stress, lb per sq in.	Cylinder No. 3 Unit strain, 10^{-5} in. per in.
3,000	106	......	107	8.0	106	8.0
6,000	213	14.5	213	17.5	213	16.5
9,000	319	21.0	320	23.5	319	......
12,000	426	28.5	427	32.0	426	34.0
15,000	532	35.5	534	......	532	......
18,000	639	44.5	640	51.0	639	52.0
21,000	745	51.0	748	......	745	61.0
24,000	852	58.5	854	67.0	852	70.0
27,000	958	65.5	961	75.0	958	79.0
30,000	1,065	72.5	1,068	84.0	1,065	87.5
33,000	1,171	81.5	1,175	94.0	1,171	97.0
36,000	1,278	90.0	1,281	103.0	1,278	105.0
39,000	1,384	98.0	1,388	113.5	1,384	115.0
42,000	1,490	106.5	1,495	124.0	1,490	126.0
45,000	1,597	115.0	1,602	136.0	1,597	136.5
48,000	1,703	125.5	1,709	145.0	1,703	147.0
51,000	1,810	134.5	1,816	157.0	1,810	157.5
54,000	1,916	144.5	1,922	167.5	1,916	172.0
57,000	2,022	153.0	2,029	177.5	2,022	186.0
60,000	2,129	164.5	2,136	193.5	2,129	199.0
66,460	......	......	......	......	2,358	Failure
67,440	......	......	2,401	Failure	......	......
71,000	2,520	Failure	......	......	......	......
0.45 max	1,134	78.0	1,080	85.0	1,061	87.5
E, lb/in.^{2}	1.45×10^6		1.27×10^6		1.21×10^6	

Data on Compression Test

Test Series No. D-4

Date: 4-14-50

Load, lb	Cylinder No. 1 6.01″ x 12.00″ Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.	Cylinder No. 2 5.98″ x 12.00″ Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.	Cylinder No. 3 6.00″ x 12.00″ Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.
3,000	106	6.5	107	6.0	106	....
6,000	212	13.0	214	12.5	212	....
9,000	317	13.0	320	18.5	318	....
12,000	423	25.5	427	25.0	424	....
15,000	529	34.0	534	32.0	531	....
18,000	635	40.5	641	38.0	637	....
21,000	740	48.0	749	44.0	743	....
24,000	846	54.5	854	51.0	849	49.0
27,000	951	61.5	961	58.0	955	56.0
30,000	1,057	68.5	1,068	64.5	1,061	62.5
33,000	1,153	75.5	1,175	71.5	1,167	68.5
36,000	1,269	83.0	1,282	78.0	1,273	75.5
39,000	1,375	90.5	1,388	84.5	1,380	82.5
42,000	1,481	96.5	1,495	91.0	1,486	79.0
45,000	1,586	103.0	1,602	98.5	1,592	96.0
48,000	1,692	....	1,709	105.0	1,698	102.5
51,000	1,798	117.5	1,816	111.5	1,804	109.0
54,000	1,904	126.5	1,922	118.0	1,910	116.0
57,000	2,009	132.5	2,029	125.0	2,016	123.0
60,000	2,115	140.5	2,136	132.0	2,122	130.5
63,000	2,221	148.5	2,243	138.5	2,229	137.0
66,000	2,327	156.5	2,350	146.0	2,335	144.0
69,000	2,432	164.0	2,456	153.0	2,441	152.0
72,000	2,538	172.5	2,563	161.0	2,547	159.0
75,000	2,644	182.5	2,670	168.0	2,653	166.0
78,000	2,750	193.0	2,777	176.0	2,759	173.5
81,000	2,855	202.0	2,884	184.0	2,865	182.0
87,340	....	....	3,109	Failure	....	....
97,100	3,423	Failure	....	....	....	....
103,040	....	....	....	....	3,645	Failure
0.45 max	1,540	99.5	1,399	85.0	1,640	99.0
E, lb/in.^{2}	1.55×10^6		1.65×10^6		1.66×10^6	

Data on Compression Test

Test Series No. C_f-4

Date: 6-17-50

Load, lb	Cylinder No. 1 5.99″ x 12.00″		Cylinder No. 2 5.98″ x 11.96″		Cylinder No. 3 5.98″ x 12.02″	
	Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.	Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.	Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.
3,000	106	8.5	107	11.0	107	6.5
6,000	213	16.0	213	20.5	213	14.0
9,000	319	24.0	320	28.0	320	22.0
12,000	426	31.5	427	31.5	427	29.0
15,000	532	39.0	534	39.0	534	37.0
18,000	639	47.0	640	46.0	640	43.5
21,000	745	54.5	748	53.0	748	51.5
24,000	852	62.0	854	61.0	854	60.5
27,000	958	70.5	961	68.5	961	68.0
30,000	1,065	78.5	1,068	76.5	1,068	76.0
33,000	1,171	86.5	1,175	83.5	1,175	84.0
36,000	1,278	94.5	1,281	91.5	1,281	92.0
39,000	1,384	103.0	1,388	99.0	1,388	99.5
42,000	1,490	112.0	1,495	107.0	1,495	108.0
45,000	1,597	121.0	1,602	116.0	1,602	116.5
48,000	1,703	129.5	1,709	125.5	1,709	126.0
51,000	1,810	139.0	1,816	133.5	1,816	136.0
54,000	1,916	148.5	1,922	143.0	1,922	144.0
76,100	2,700	Failure	....	....	....	....
77,160	....	....	2,747	Failure	....	....
78,500	....	....	....	....	2,795	Failure
0.45 max	1,215	89.5	1,236	88.5	1,258	90.0
E, lb/in.^{2}	1.36×10^6		1.40×10^6		1.40×10^6	

Data on Compression Test

Test Series No. D_f-4

Date: 6-10-50

Load, lb	Cylinder No. 1 6.00" x 12.00" Unit stress, lb per sq in.	Cylinder No. 1 Unit strain, 10^{-5} in. per in.	Cylinder No. 2 5.98" x 11.96" Unit stress, lb per sq in.	Cylinder No. 2 Unit strain, 10^{-5} in. per in.	Cylinder No. 3 5.97" x 12.00" Unit stress, lb per sq in.	Cylinder No. 3 Unit strain, 10^{-5} in. per in.
3,000	106	6.5	107	6.0	107	6.5
6,000	212	12.5	214	12.0	214	13.0
9,000	318	17.5	320	18.0	322	19.0
12,000	424	23.0	427	24.0	429	25.0
15,000	531	29.0	534	30.0	536	31.5
18,000	637	35.0	641	35.5	643	37.5
21,000	743	41.0	748	41.5	750	43.0
24,000	849	47.0	854	47.5	858	49.0
27,000	955	53.0	961	54.0	965	56.0
30,000	1,061	58.5	1,068	60.0	1,072	62.0
33,000	1,167	64.5	1,176	65.5	1,179	68.5
36,000	1,273	70.0	1,282	71.5	1,286	74.5
39,000	1,380	76.0	1,388	77.5	1,394	81.0
42,000	1,486	82.0	1,495	83.0	1,501	86.5
45,000	1,592	87.5	1,602	89.0	1,608	92.5
48,000	1,698	94.5	1,709	95.5	1,715	98.0
51,000	1,804	98.5	1,816	101.5	1,822	104.0
54,000	1,910	105.5	1,922	108.0	1,930	111.5
57,000	2,016	111.5	2,029	114.5	2,037	117.5
60,000	2,122	118.0	2,136	120.5	2,144	124.0
63,000	2,229	124.0	2,243	126.5	2,251	131.0
66,000	2,335	130.0	2,350	133.0	2,358	137.5
69,000	2,441	136.0	2,456	140.0	2,466	143.5
72,000	2,547	143.0	2,563	146.0	2,573	150.5
75,000	2,653	150.0	2,670	152.5	2,680	157.5
78,000	2,759	156.5	2,777	159.5	2,787	164.0
81,000	2,865	163.0	2,884	166.0	2,894	172.5
84,000	2,971	170.0	2,990	172.5	3,002	179.0
115,980	....	....	....	....	4,144	Failure
119,440	4,225	Failure	....	....	....	....
121,020	....	....	4,315	Failure	....	....
0.45 max	1,901	105.5	1,942	108.5	1,865	107.0
E, lb/in.^{2}	1.80×10^6		1.79×10^6		1.74×10^6	

Data on Compression Test

Test Series No. E-4

Date: 4-25-50

Load, lb	Cylinder No. 1 5.99″ x 11.96″ Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.	Cylinder No. 2 5.99″ x 11.97″ Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.	Cylinder No. 3 5.99″ x 12.00″ Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.
3,000	106	8.0	106	9.0	106	8.5
6,000	213	16.5	213	17.0	213	17.5
9,000	319	25.5	319	26.0	319	25.5
12,000	426	34.0	426	34.0	426	33.5
15,000	532	43.0	532	42.5	532	41.5
18,000	639	51.0	639	51.5	639	50.0
21,000	745	60.0	745	60.0	745	59.0
24,000	852	69.5	852	69.0	852	67.5
27,000	958	78.0	958	77.5	958	76.5
30,000	1,065	88.0	1,065	86.0	1,065	85.5
33,000	1,171	98.0	1,171	95.5	1,171	94.0
36,000	1,278	107.0	1,278	104.0	1,278	102.5
39,000	1,384	117.0	1,384	113.5	1,384	112.5
42,000	1,490	128.0	1,490	124.0	1,490	122.0
45,000	1,597	138.5	1,597	133.0	1,597	131.0
56,000	1,987	....	....	....	....	....
66,260	....	....	....	....	2,351	....
68,120	....	....	2,417	....	....	....
0.45 max ..	894	73.0	1,088	88.5	1,058	83.5
E, lb/in.^{2} ..	1.22×10^6		1.23×10^6		1.27×10^6	

Data on Compression Test

Test Series No. G-4

Date: 4-19-50

Load, lb	Cylinder No. 1 5.99″ x 12.00″		Cylinder No. 2 5.98″ x 12.08″		Cylinder No. 3 6.00″ x 12.06″	
	Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.	Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.	Unit stress, lb per sq in.	Unit strain, 10^{-5} in. per in.
3,000	106	3.5	107	2.5	106	2.0
6,000	213	7.0	214	6.0	212	5.0
9,000	319	10.0	320	9.0	318	7.5
12,000	426	13.5	427	12.5	424	11.0
15,000	532	17.0	534	15.5	531	13.5
18,000	639	20.0	641	18.5	637	16.5
21,000	745	23.5	748	21.5	743	19.5
24,000	852	27.0	854	25.0	849	22.5
27,000	958	30.5	961	28.0	955	26.0
30,000	1,065	34.0	1,068	31.5	1.061	29.0
33,000	1,171	37.5	1,175	34.5	1,167	32.0
36,000	1,278	41.5	1,282	38.0	1,273	36.0
39,000	1,384	45.0	1,388	41.5	1,380	39.0
42,000	1,490	48.5	1,495	44.0	1,486	42.5
45,000	1,597	52.5	1,602	48.0	1,592	46.0
48,000	1,703	56.5	1,709	51.5	1.698	49.0
51,000	1,810	60.0	1,816	55.5	1,804	53.0
54,000	1,916	65.0	1,922	58.5	1,910	57.0
57,000	2,023	68.0	2,029	62.5	2,016	61.0
60,000	2,129	73.5	2,136	67.5	2,122	64.0
63,000	2,226	77.5	2,243	72.0	2,229	68.5
66,000	2,342	83.0	2,350	77.0	2,335	73.5
69,000	2,449	88.5	2,456	82.0	2,441	78.5
72,000	2,555	93.5	2,563	87.0	2,547	83.5
93,890	....	....	3,342	Failure	....	....
94,830	....	....	....	....	3,354	Failure
97,290	3,452	Failure	....	....	....	....
0.45 max	1,553	50.5	1,504	45.5	1,509	43.0
E, lb/in.^{2}	3.08×10^6		3.31×10^6		3.51×10^6	

Data on Compression Test

Test Series No. H-4

Date: 4-18-50

Load, lb	Cylinder No. 1 5.99″ x 12.04″ Unit stress, lb per sq in.	Cylinder No. 1 Unit strain, 10^{-5} in. per in.	Cylinder No. 2 5.99″ x 12.02″ Unit stress, lb per sq in.	Cylinder No. 2 Unit strain, 10^{-5} in. per in.	Cylinder No. 3 6.00″ x 12.00″ Unit stress, lb per sq in.	Cylinder No. 3 Unit strain, 10^{-5} in. per in.
3,000	106	5.5	106	4.0	106	5.0
6,000	213	10.0	213	9.0	212	10.5
9,000	319	15.0	319	13.5	318	15.0
12,000	426	20.0	426	19.0	424	20.0
15,000	532	25.5	532	24.0	531	25.0
18,000	639	30.5	639	29.0	637	30.0
21,000	745	35.5	745	34.0	743	35.0
24,000	852	41.0	852	38.0	849	39.5
27,000	958	46.5	958	43.0	955	44.5
30,000	1,065	52.0	1,065	48.0	1,061	49.5
33,000	1,171	57.0	1,171	52.5	1,167	54.5
36,000	1,278	62.5	1,278	57.5	1,273	60.0
39,000	1,384	68.0	1,384	62.5	1,380	65.0
42,000	1,490	73.5	1,490	68.0	1,486	70.0
45,000	1,597	80.0	1,597	73.5	1,592	75.5
48,000	1,703	85.0	1,703	79.0	1,698	80.5
51,000	1,810	90.5	1,810	84.5	1,804	85.5
54,000	1,916	97.5	1,916	90.5	1,910	90.5
57,000	2,022	103.5	2,022	95.5	2,016	96.0
60,000	2,129	110.5	2,129	102.0	2,122	102.0
63,000	2,236	116.5	2,236	108.0	2,229	108.0
66,000	2,342	123.5	2,342	113.5	2,335	115.0
69,000	2,449	131.5	2,449	120.0	2,441	121.0
72,000	2,555	139.5	2,555	127.5	2,547	128.5
88,270	3,132	Failure	....	....	....	....
104,150	....	....	....	....	3,684	Failure
109,640	....	....	3,891	Failure	....	....
0.45 max	1,409	69.0	1,751	....	1,658	78.5
E, lb/in.^{2}	2.04×10^6		2.15×10^6		2.11×10^6	

Data on Ninety-Day Compressive Strength Test

Test Series No. 5

Specimen: 4″ x 8″ cylinders

Curing: 7 days moist, 83 days air

Mix and specimen number	Date tested	Dimensions, in.		90 day weight, lb	Ultimate load, lb	Type of break	Per cent broken aggregate	f'_c, lb per sq in.
		Diameter	Height					
Mix A								
1	6–23–50	3.95	8.00	3.72	14,510	Diagonal	25	1,180
2	6–23–50	3.97	8.08	3.80	18,080	Diagonal	35	1,460
3	6–23–50	3.97	8.06	3.86	18,040	Diagonal	35	1,460
Mix B								
1	6–21–50	4.00	8.15	4.27	27,640	Diagonal	80	2,200
2	6–21–50	3.99	8.06	4.22	22,880	Diagonal	80	1,830
3	6–21–50	3.95	8.06	4.21	27,530	Diagonal	90	2,250
Mix C								
1	6–16–50	3.99	8.10	4.23	29,790	Cone	80	2,380
2	6–16–50	3.98	8.00	4.18	31,760	Diagonal	----	2,550
3	6–16–50	3.96	8.10	4.23	31,280	Diagonal	----	2,540
Mix D								
1	6–15–50	4.01	8.06	4.55	32,940	Cone	75	2,610
2	6–15–50	3.97	8.12	4.51	35,010	Cone	----	2,830
3	6–15–50	4.00	8.12	4.89	45,400	Diagonal	----	3,610
Mix E								
1	6–26–50	3.98	8.10	4.44	34,770	Diagonal	80	2,800
2	6–26–50	3.99	8.08	4.47	32,630	Diagonal	80	2,616
3	6–26–50	3.99	8.10	4.41	36,990	Diagonal	80	2,960
Mix G								
1	6–20–50	3.97	8.05	7.92	35,120	Cone	----	2,840
2	6–20–50	4.00	8.04	8.03	37,000	Diagonal	1	2,940
3	6–20–50	4.01	8.06	7.74	35,990	Diagonal	1	2,850
Mix H								
1	6–19–50	4.00	8.02	5.55	38,040	Diagonal	35	3,030
2	6–19–50	4.00	8.08	5.60	40,570	Diagonal	----	3,230
3	6–19–50	4.01	8.06	5.67	37,530	Diagonal	----	2,970

Data on Flexure Test

Test Series No. 6

Specimens: 6″ x 6″ x 36″ beams   Curing: 28 days moist

	Mix A	Mix B	Mix C	Mix D	Mix C_f	Mix D_f	Mix E	Mix G	Mix H
Break No. 1..	210	330	390	430	480	530	450	450	500
Break No. 2..	190	330	410	450	500	500	460	470	510

Data on Sonic Modulus Test

Test Series No. 6_s

Specimens: 6″ x 6″ x 36″ beams   Curing: 28 days moist

	Date tested	Depth, in.	Width, in.	Weight, lb	"Range"* of test	Dial reading	Frequency, Cycles/sec	E_s, 10^6 lb/in.^{2}
Mix A ..	4–22–50	6.00	6.00	59.0	2	45.2	455	1.29
Mix B ..	4–20–50	6.00	6.00	60.0	2	42.0	500	1.59
Mix C ..	4–15–50	6.05	6.00	58.5	2	41.0	512	1.58
Mix D ..	4–14–50	6.00	5.90	65.0	2	38.5	549	2.11
Mix C_f..	6–17–50	6.00	6.00	61.6	2	39.0	540	1.90
Mix D_f..	6–10–50	5.90	5.90	62.3	2	39.0	540	2.06
Mix E ..	4–25–50	6.00	6.00	61.0	2	41.6	503	1.64
Mix G ..	4–19–50	6.00	5.90	106.0	2	33.3	640	4.65
Mix H ..	4–18–50	5.95	5.90	74.5	2	36.6	580	2.77

* Each "range" corresponds to a certain range of frequencies and is selected on the sonic modulus tester by the setting of a panel-board knob.

Data on Bond Test

Test Series No. 7

Specimens: 8″ x 8″ cylinders with ⅝″ round deformed bars

Curing: 7 days moist, 21 days air

Mix and specimen number	Date tested	Height, in.	Load at end slip, lb	Load at failure, lb	Type of failure	Bond area, in.^{2}	Average bond stress, lb/in.^{2}	
							End slip	Failure
Mix A								
1	4–22–50	8.00	4,120	9,390	Split	15.71	262	598
2	4–22–50	8.08	4,000	8,310	Split	15.87	252	524
3	4–22–50	8.00	4,000	7,430	Pull out	15.71	255	473
Mix B								
1	4–20–50	8.08	5,490	10,360	Split	15.87	346	653
2	4–20–50	8.08	5,400	8,690	Split	15.87	340	548
3	4–20–50	8.02	5,800	9,690	Split	15.75	368	615
Mix C								
1	4–15–50	8.04	9,120	11,440	Split	15.79	578	725
2	4–15–50	8.08	8,130	10,870	Split	15.87	512	685
3	4–15–50	8.10	7,850	12,340	Split	15.91	493	776
Mix D								
1	4–14–50	8.06	7,480	9,440	Split	15.83	473	596
2	4–14–50	8.08	8,650	14,010	Split	15.87	545	883
3	4–14–50	8.05	8,720	13,480	Split	15.81	552	853
Mix C_f								
1	6–17–50	8.02	5,240	9,670	Split	15.75	333	614
2	6–17–50	8.00	7,210	10,920	Split	15.71	459	695
3	6–17–50	8.00	7,630	12,430	Split	15.71	486	791
Mix D_f								
1	6–10–50	7.95	9,150	14,460	Split	15.61	586	926
2	6–10–50	7.98	8,350	15,670	Split	15.67	533	1,000
3	6–10–50	8.15	9,560	9,560	Split	16.01	597	597
Mix E								
1	4–25–50	8.06	7,320	9,870	Split	15.83	462	623
2	4–25–50	8.04	6,410	10,350	Split	15.79	406	655
3	4–25–50	8.10	4,880	9,410	Split	15.91	307	591
Mix G								
1	4–19–50	8.16	4,930	17,150	Split	16.03	308	1,070
2	4–19–50	8.05	4,120	19,020	Steel	15.81	261	1,203
3	4–19–50	8.01	6,010	19,110	Steel	15.73	382	1,215
Mix H								
1	4–18–50	8.02	9,370	19,070	Steel	15.75	618	1,211
2	4–18–50	8.04	7,420	18,950	Pull out	15.79	470	1,200
3	4–18–50	8.06	8,850	17,740	Split	15.83	559	1,121

Data on Absorption Test

Test Series No. 9

Specimens: 4" x 8" cylinders

Curing: 7 days moist, 21 days air

Mix and specimen number	Dimensions, in. Diam	Dimensions, in. Height	Oven dry wt, lb	Oven dry unit wt, lb/cu ft	24-hour immersion wt, lb	Absorption, per cent By dry wt	Absorption, per cent By volume
Mix A							
1	4.00	8.06	3.63	62.1	4.31	18.7	18.7
2	*	*	3.51	......	4.20	19.6	......
3	3.99	8.08	3.59	61.5	4.27	18.9	18.7
Mix B							
1	3.97	8.10	3.78	65.2	4.27	13.0	13.5
2	3.98	8.06	3.73	64.4	4.23	13.4	13.8
3	4.00	8.04	3.74	64.0	4.25	13.6	14.0
Mix C_f							
1	4.01	8.12	4.12	69.5	4.67	13.4	14.9
2	3.98	8.04	4.20	72.5	4.74	12.9	15.0
3	3.99	8.05	4.11	70.5	4.65	13.1	14.9
Mix D_f							
1	4.00	8.01	4.43	75.8	4.95	11.7	14.3
2	3.96	8.06	4.43	77.1	4.93	11.3	14.0
3	4.00	8.06	4.44	75.9	4.95	11.5	14.0
Mix E							
1	3.99	8.06	3.96	67.9	4.55	14.9	16.2
2	4.00	8.08	3.98	67.7	4.56	14.6	15.8
3	3.98	8.00	3.85	66.9	4.44	15.3	16.4
Mix G							
1	4.00	8.06	7.91	134.9	8.41	6.3	13.7
2	3.99	8.14	8.09	137.4	8.53	5.4	12.0
3	4.00	8.06	8.01	136.6	8.47	5.7	12.6
Mix H							
1	4.00	8.03	5.06	87.7	5.62	11.1	15.4
2	3.98	8.02	5.09	86.7	5.62	11.0	15.5
3	4.02	7.99	5.08	86.6	5.63	10.9	15.0

* Poor surface.

Data on Shrinkage Test

Test Series No. 10

Specimens: 3″ x 3″ x 11″ bars Curing: 7 days moist, 21 days air

Mix and specimen number	Length, in.			Shrinkage, per cent	
	1 day	28 days	Oven dry	28-day average	Oven-dry average
Mix A					
1	10.0270	10.0233	10.0209	0.027	0.047
2	10.0181	10.0154	10.0139	....	....
3	9.9960	9.9943	9.9922	....	....
Mix B					
1	10.0317	10.0288	10.0267	0.029	0.053
2	9.9978	9.9950	9.9927	....	....
3	10.0041	10.0010	9.9983	....	....
Mix C_f					
1	10.0009*	9.9961	9.9935	0.036	0.068
2	10.0004*	9.9976	9.9941	....	....
3	10.0030*	9.9997	9.9962	....	....
Mix D_f					
1	9.9931	9.9912	....	0.027	....
2	10.0154	10.0126	....	....	....
3	10.0079	10.0043	....	....	....
Mix E					
1	9.9957	9.9927	9.9894	0.029	0.061
2	9.9959	9.9931	9.9899	....	....
3	10.0006	9.0078	9.9946	....	....
Mix G					
1	10.0028	9.9986	9.9934	0.046	0.094
2	10.0036	9.9986	9.9947	....	....
3	10.0078	10.0031	9.9980	....	....
Mix H					
1	10.0071*	10.0032	....	0.057	....
2	10.0035*	9.9968	....	....	....
3	10.0038*	9.9973	....	....	....

* Unstable on first day, measured at approximately 36 hours.

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