"LITE ROCK" IN STRUCTURAL CONCRETE

by

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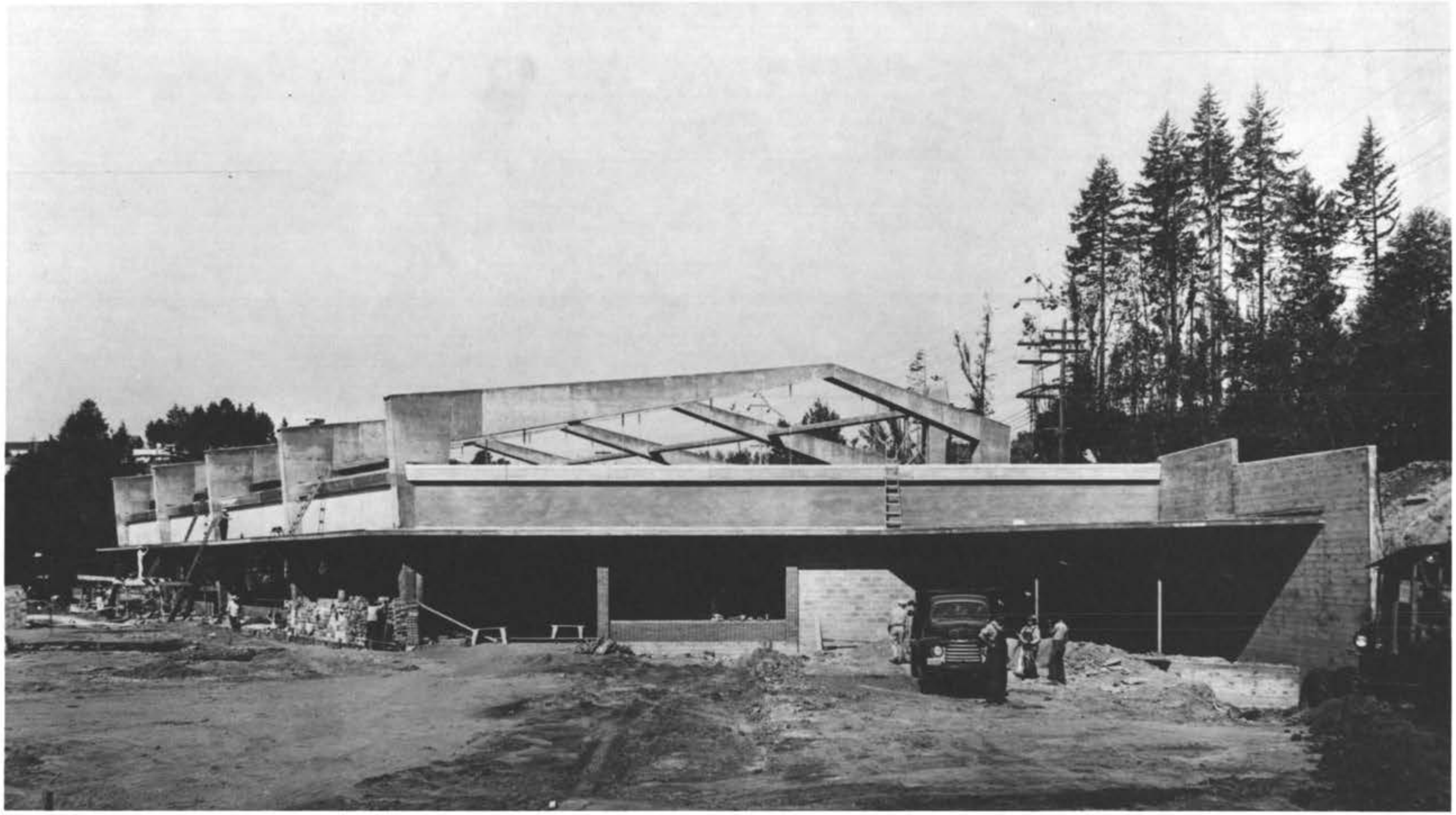

Figure 1. Fred Meyer Shopping Center, Portland, Oregon,

with trusses of Lite Rock Concrete.

"LITE ROCK" IN STRUCTURAL CONCRETE

INTRODUCTION

1. Applications of lightweight concrete. The use of lightweight concrete is not new, 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 lightweight concrete are in use. Probably the most notable example is the placing of "Gravelite" lightweight-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 in Portland, Oregon, (Fig. 1) where "Lite Rock" aggregate concrete, which is to be the subject of this paper, is being used. Here a floor one hundred and thirty feet in clear-span width is achieved by lightweight concrete trusses.

2. Need for design information. With the expanding use of lightweight aggregate concrete a demand arises for information descriptive of its behavior. Architects, engineers, contractors, and builders, desiring to use lightweight 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 lightweight concrete. Recognition of this fact has resulted in the publication, "Lightweight Aggregate Concretes," (1, p.II) recently issued by the Housing and Home Finance Agency. This publication shows not only that these aggregates differ from sand and gravel, but that wide variations may be expected between different types of lightweight aggregate and that each particular aggregate requires individual study.

It was with the object of securing 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

TABLE I

OUTLINE OF PRINCIPAL TESTS

Test		Specimens										
				Number								
				Mix								
Name	No.	Size	Curing	A	B	C	D	C_f	D_f	E	G	H
Compressive Strength	1	4" x 8" cyl.	7 day moist	3	3	3	3	3	3	3	3	3
	2	4" x 8" cyl.	7 day moist 21 day air	3	3	3	3	3	3	3	3	3
	3	4" x 8" cyl.	28 day moist	3	3	3	3	3	3	3	3	3
(Mod. of Elasticity)	4	6" x 12" cy.	28 day moist	3	3	3	3	3	3	3	3	3
	5	4" x 8" cyl.	7 day moist 83 day air	3	3	3	3			3	3	3
Flexure	6	6" x 6" x 36"	28 day moist	1	1	1	1	1	1	1	1	1
Sonic Modulus	6_s											
Bond	7	8" x 8" cyl.	7 day moist 21 day air	3	3	3	3	3	3	3	3	3
Dorry Abrasion	8	2" x 4" cyl.	7 day moist 21 day air	1	1			1	1	1	1	1
Absorption	9	4" x 8" cyl.	7 day moist 21 day air	3	3			3	3	3	3	3
Shrinkage	10	3" x 3" x 11"	7 day moist 21 day air	3	3			3	3	3	3	3

TABLE I
(Cont'd.)

OUTLINE OF PRINCIPAL TESTS

Nine mixes were used as follows:

Aggregate: A,B,C,D,C_f,D_f,E-Lite Rock: G-Gravel: H-Expanded Shale No. 2. All dry batched. Maximum size: A,B-3/4"; C,D,E,H,-3/8"; C_f,D_f-1/4"; G-1".

Cement Factor, sk. cu. yd.: A-3.7; B-5.4; C-6.9; D-9.2; C_f-6.9; D_f-8.8; E-7.1; G-4.8; H-6.9.

Dispersing Agent: 1/2 lb. per sack cement in all but mix E.

Water: Sufficient to provide good workability.

Banks, Oregon. The burning is accomplished in a rotary kiln at temperatures in excess of 2200 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 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 Lite Rock are dealt with briefly in Part I.

4. Outline and scope of work. The investigation considered here consists primarily of tests on Lite Rock 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 lightweight concretes are shown in Figures 2 and 3. The materials used in the tests are described in Part II, and their proportioning and mixing in Part III. The concrete tests are outlined in Table I, described in Part IV,

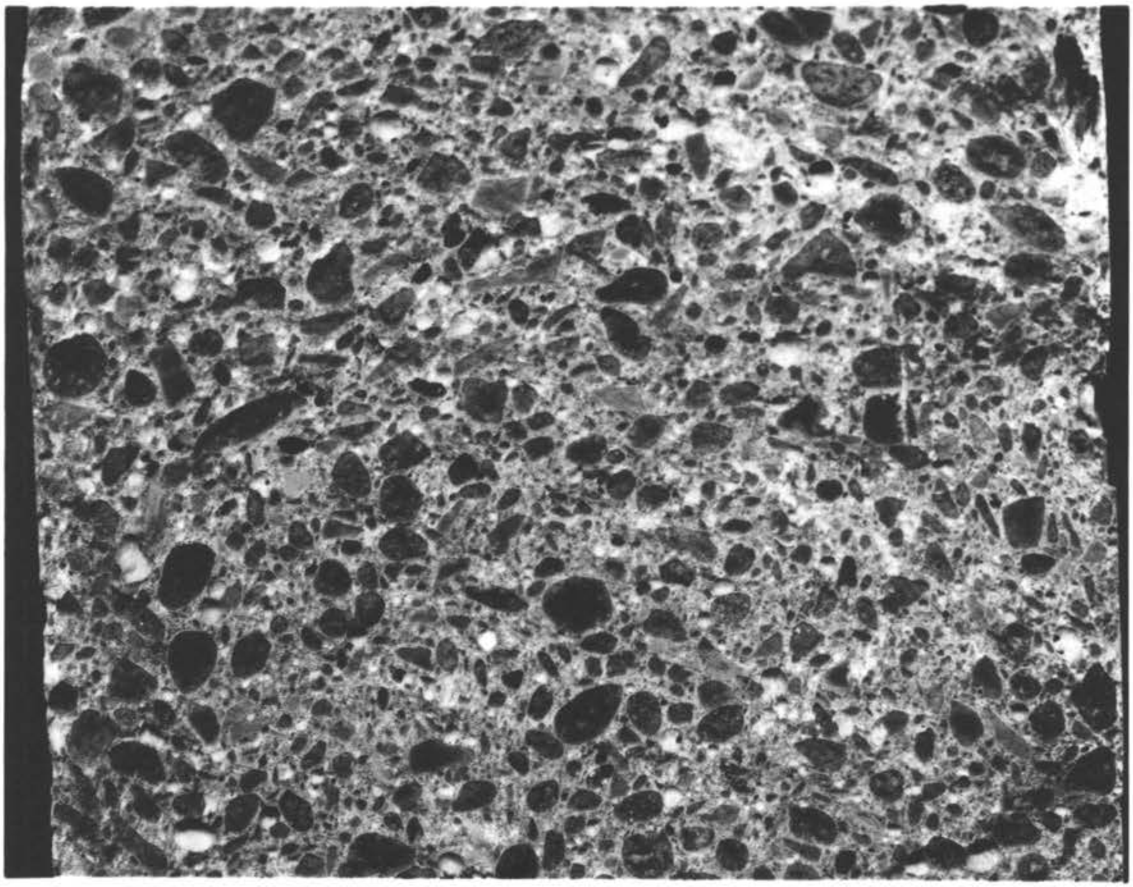

Figure 2. Section Through Lite Rock Concrete (Actual Size).

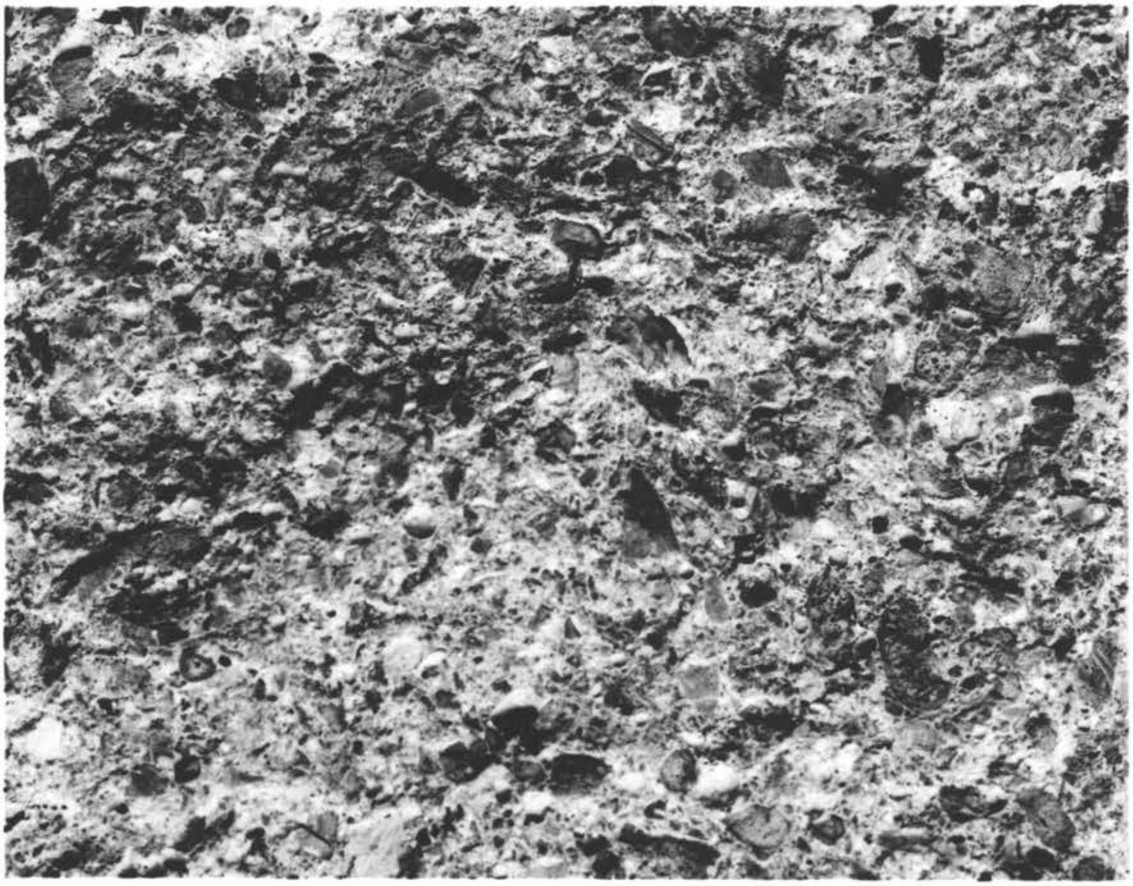

Figure 3. Section Through Expanded-Shale No. 2
Concrete (Actual Size).

and furnish material for the discussion and design data taken up in Parts V and VI.

The testing program was arranged to facilitate comparison with the extensive work done on lightweight aggregate concretes by the Bureau of Reclamation and the National Bureau of Standards which is reported in "Lightweight Aggregate Concretes" (1). Cement factors were chosen in the neighborhood of three, five, seven, and nine 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 gravel were used. The mixes are taken up in detail in Part III.

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.

$K = 1/2\, f_c kj = pf_s j$.

$n = \frac{E_s}{E_c}$ = 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.

PART I-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 stated in Part VI. 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 course, Structural Materials Lab. Five beams were tested, two with crushed-Lite Rock aggregate and three with 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.

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

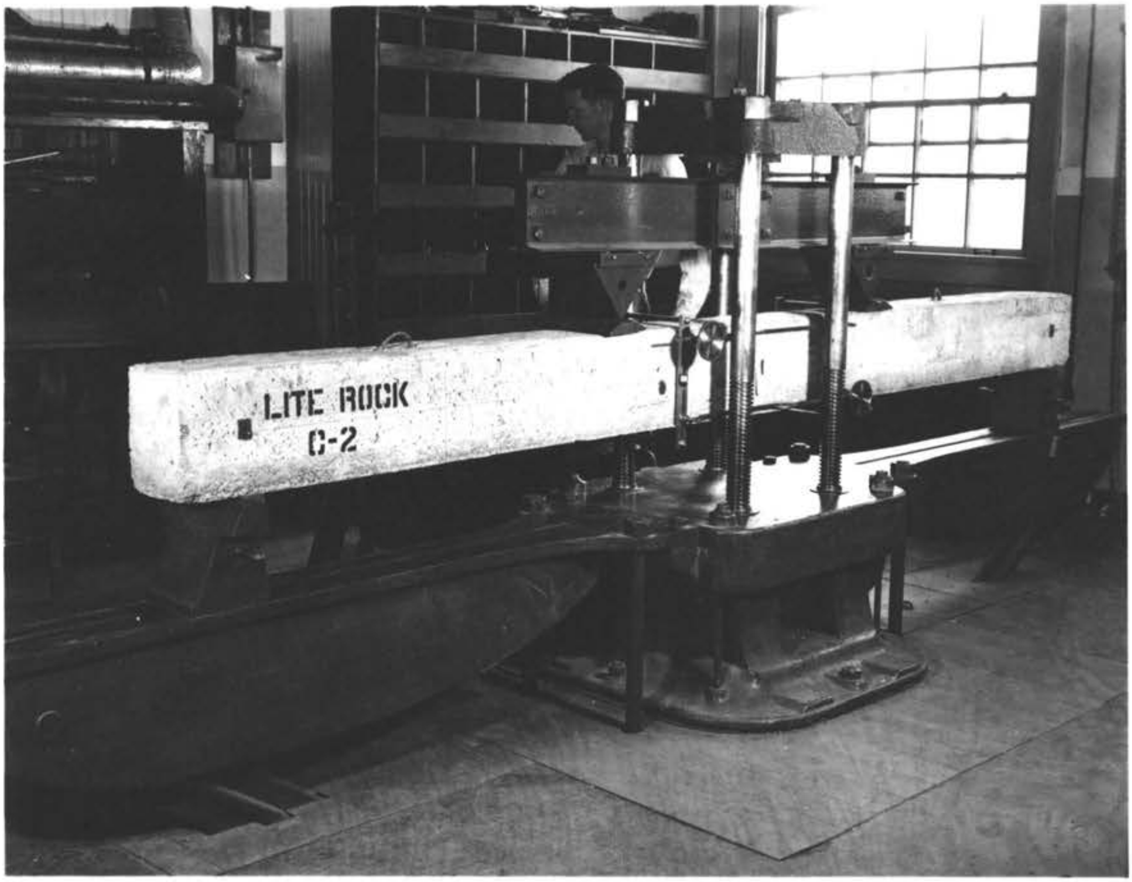

Figure 4. Beam Test on Crushed-Lite Rock Concrete.

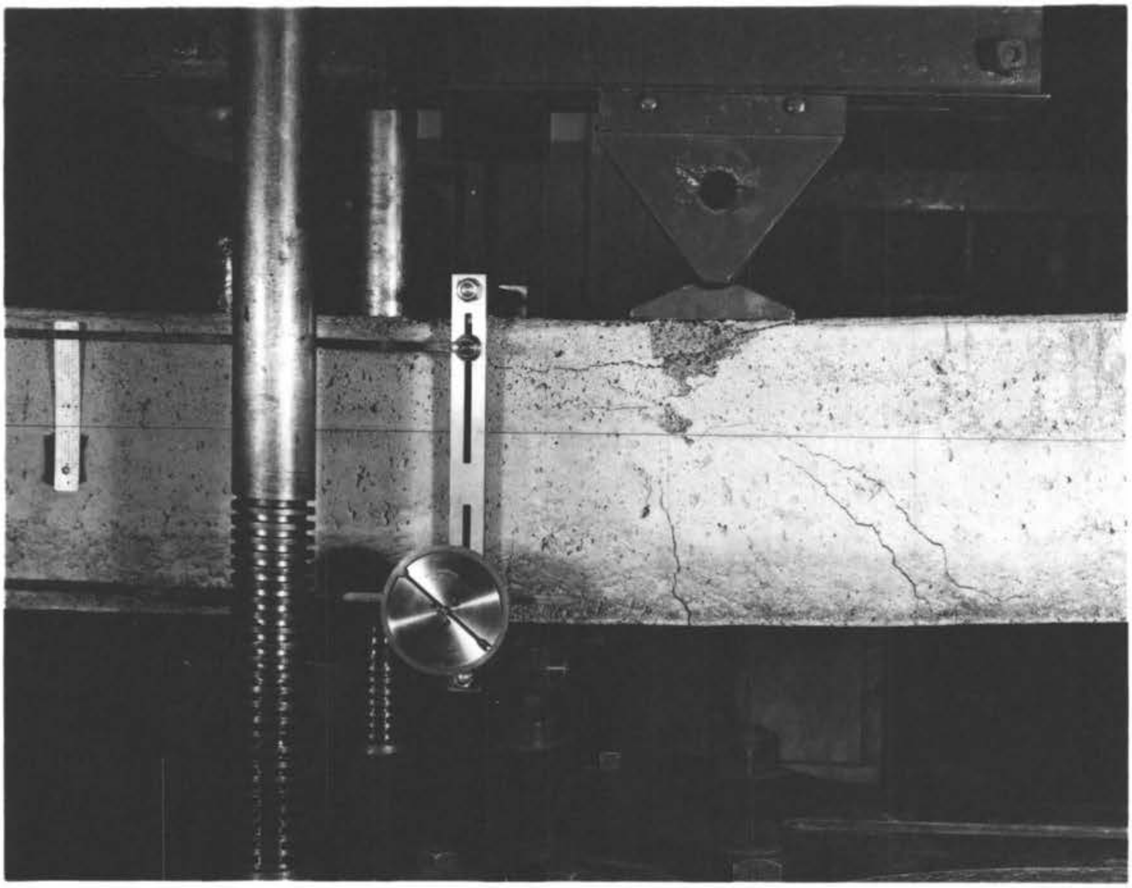

Figure 5. Failure of Crushed-Lite Rock Beam Due to Tension in Steel.

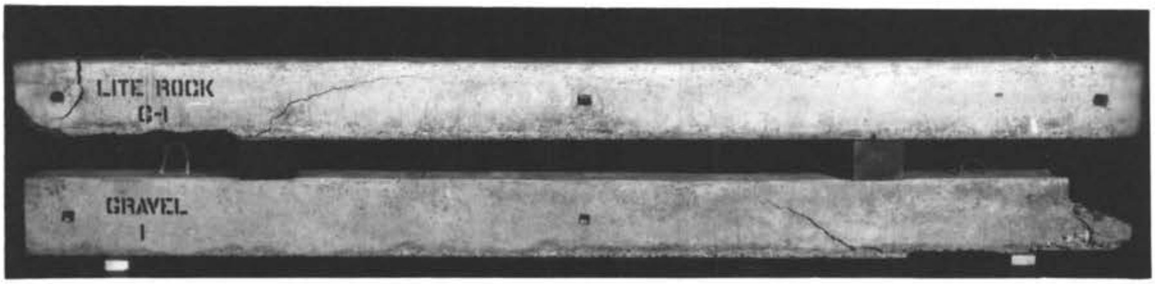

Figure 6. Beams Without Stirrups, After Test.

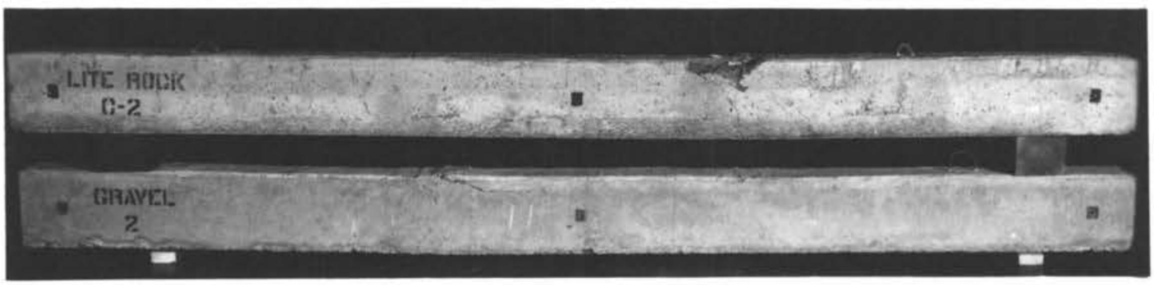

Figure 7. Beams With Stirrups, After Test.

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 lightweight 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 (Fig. 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 IV of this paper.

4. Beams with web reinforcing. The two beams with stirrups (Fig. 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 relative stresses in Lite Rock and gravel concrete

beams for equal loads. This will be discussed in Part VI where design of Lite Rock concrete is considered.

PART II-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 test was structural grade.

2. Description of the aggregates. Coarse and fine Lite Rock aggregate is pictured in Figures 8 and 9. This aggregate is 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 aggregate is not as smooth as natural gravel, but much less harsh than a crushed stone, or a shale which has been crushed after expansion. The glaze coating also furnishes 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 as does the crushed material.

Figure 8. Coarse Lite Rock Aggregate
(Actual Size).

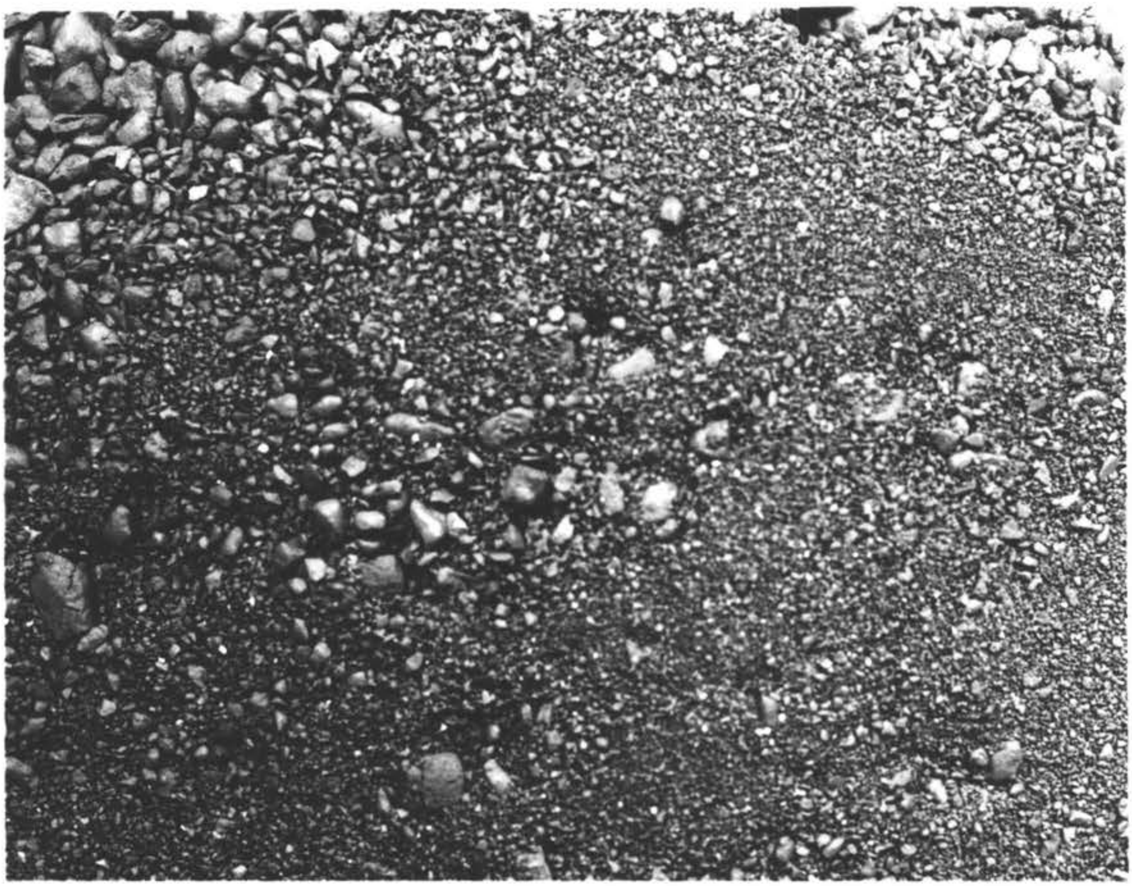

Figure 9. Fine Lite Rock Aggregate

(Actual Size).

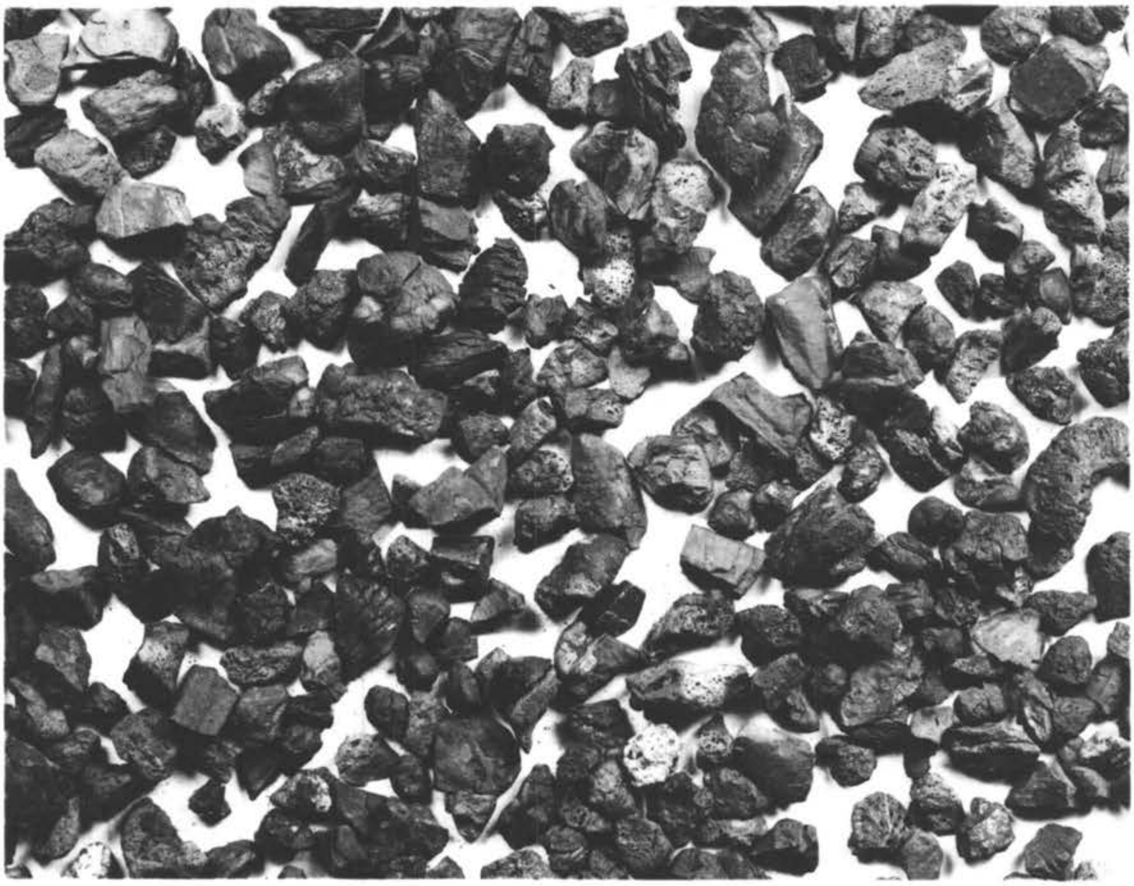

Figure 10. Coarse Expanded-Shale No. 2 Aggregate (Actual Size).

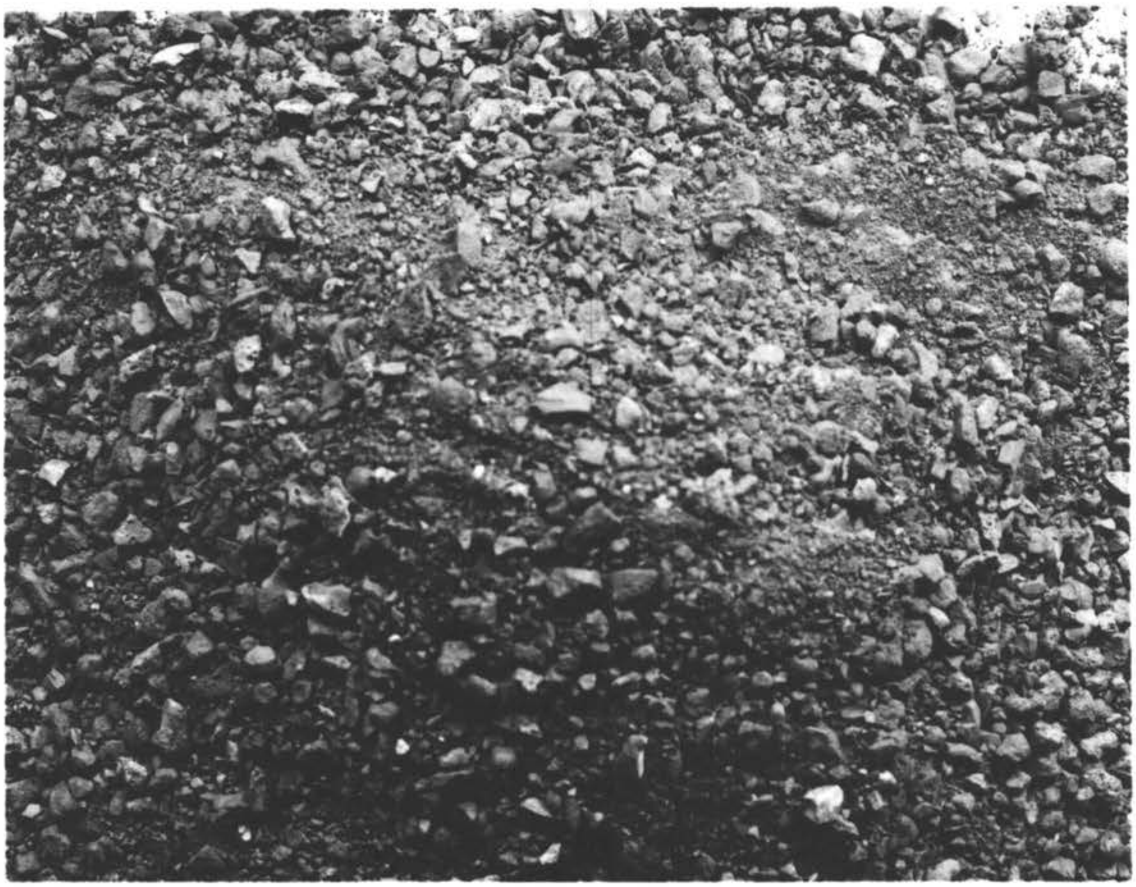

Figure 11. Fine Expanded-Shale No. 2 Aggregate (Actual Size).

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 partially recrushed as shown in the photograph. However, much of it was coated and it differed from Lite Rock principally by its greater weight.

Columbia-River sand and gravel were obtained from 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 II 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 need 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 1/4 cubic foot measure of the aggregate rodded as described in A.S.T.M. Designation: C 29-42.

TABLE II
SIEVE ANALYSES OF AGGREGATES

Aggregate	Mix	Per cent by Weight Retained on Tyler Sieve No.								Fineness Modulus
		3/4"	3/8"	4	8	14	28	48	100	
<u>Lite Rock</u>										
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_f	--	--	7	25	46	70	92	100	3.40
Combined	D_f	--	--	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
<u>Gravel</u>										
Coarse	G	31	78	93	95	96	97	99	100	6.89
Fine	G	--	--	7	26	41	59	89	99	3.21
<u>Expanded Shale No. 2</u>										
Coarse	H	--	--	84	99	99	99	99	100	5.80
Fine	H	--	--	--	29	56	76	87	93	3.41

The unit weights of the aggregates, along with other physical properties, are listed in Table III. 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 A.S.T.M. Designation: C 128-42 as far as possible. In other lightweight-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 aggregate immersed. The fine lightweight aggregates were considered saturated-surface-dry when they would flow freely through the fingers

TABLE III
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_f	48.6	0.0			
Combined	D_f	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

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 aggreement 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 III.

PART III-PROPORTIONING AND MIXING

A summary of mix data is given in Table IV, and the data are tabulated completely in the Appendix.

1. Maximum size. Proportioning of Lite Rock aggregate is complicated by the 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 lightweight-aggregate concrete is a direct function of the aggregate strength.

With this in mind, 3/4-inch aggregate was used in the two leaner mixes, A and B, while 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 and nine sack mixes, C_f and D_f, were made with a maximum aggregate size of 1/4-inch.

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 3/8-inch as furnished from the plant.

TABLE IV
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	3/4"	3/4"	3/8"	3/8"	1/4"	1/4"	3/8"	1"	3/8"
Per cent Coarse, by wt.	20	30	32	35	--	--	20	55	26
Dispersing Agent	Yes	Yes	Yes	Yes	Yes	Yes	No	Yes	Yes
Water-cement ratio, by. wt.	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 wt., lb./cu. ft.	76.4	79.9	75.2	84.8	83.0	86.5	80.3	143.8	99.9

2. Proportions. After deciding upon 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 lightweight 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 IV 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 lightweight aggregates because of high absorption and varying rate of absorption with different 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 1 1/2-cubic foot tilt-drum mixer. While 1 1/2-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 one cubic foot or less. The comparison shale was also mixed in the smaller batches.

The lightweight 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 five minutes for all mixes except the two leanest, A and G, which were mixed eight and ten 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 four 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 lightweight concrete: There is less weight to overcome cohesive forces and cause slump. 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 six-inch cylinders and other large specimens. It was used in the same way in the measuring bucket which served to determine unit weights and cement factors.

For the four-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 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.

PART IV-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 cure as was one set of 4" x 8" cylinders for comparison. All other specimens were given only a seven day moist cure to correspond more closely with job practice. The remainder of the curing was accomplished in room air.

The moist curing was effected in a fog room at 100 per cent humidity and 70 F. For the "air" curing, the specimens were placed in a curing room which remained at approximately 50 per cent humidity and 70 F.

2. Compressive strength tests (No. 1-5). Compressive strength tests were made at seven, twenty-eight, and ninety days. Three tests were made at the twenty-eight day age to furnish a comparison of curing condition effects, and a comparison between strength of four-inch and six-inch cylinders. Results of compressive strength tests are summarized in Table V, and complete data are given in the Appendix.

TABLE V

RESULTS OF COMPRESSIVE STRENGTH TESTS
(TEST NOS. 1-5)

Mix	Cement Factor	Water-Cement Ratio, by Wt.	Slump, in.	Unit Weight, lb./cu.ft.		Compressive Strength lb./sq.in.					Ratio, Strength-Weight, psi/lb.
						4" x 8" Cylinders				6"x12" cyl.	
				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	1200	1060	1370	1200	15.9
B	5.4	0.68	2.3	79.9	64.5	1670	2050	1960	2090	2170	27.2
C	6.9	0.49	3.0	75.2		2270	2670	2590	2490	2430	32.3
D	9.2	0.40	5.0	84.8		2810	2860	2890	3020	3390	40.0
C_f	6.9	0.64	4.6	83.0	70.8	2180	2880	2720		2750	33.2
D_f	8.8	0.47	5.3	86.5	76.3	3390	3480	3770		4220	48.8
E	7.1	0.61	1.8	80.3	67.5	1980	2500	2480	2790	2250	28.0
G	4.8	0.61	5.3	143.8	136.3	2030	3100	3160	2880	3380	24.8
H	6.9	0.55	2.1	99.9	87.0	1830	3080	2900	3080	3570	35.7

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

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 inches 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 ten lb./sq. in.

3. Compression Test (No. 4). A test for modulus of elasticity was made on all of the six-inch cylinders. The apparatus used was the Graf strainometer with a dial gage reading to 0.001 inches. This device, set up on a specimen at a ten 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 inches per minute. The load was applied in 3000 pound increments and gage readings were made at each increment. This was continued until approximately two-thirds of the ultimate load was reached. The apparatus was then removed and the specimen loaded until failure. Stress-strain curves are shown in Figures 13, 14, and 15, and values for the secant modulus of elasticity, taken at 0.45 f'_c are plotted in Figure 16. Complete data for the compression test are included in the Appendix.

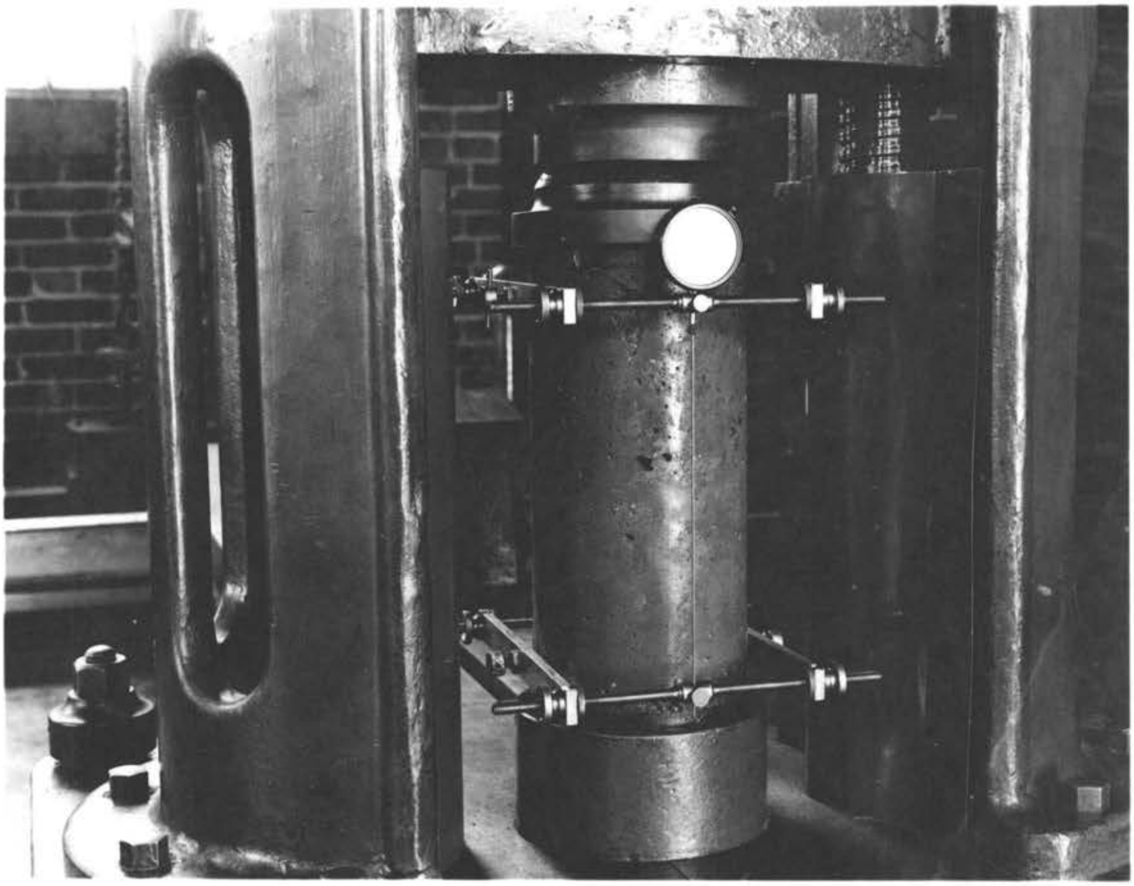

Figure 12. Compression Test Cylinder
set up with Graf Strainometer.

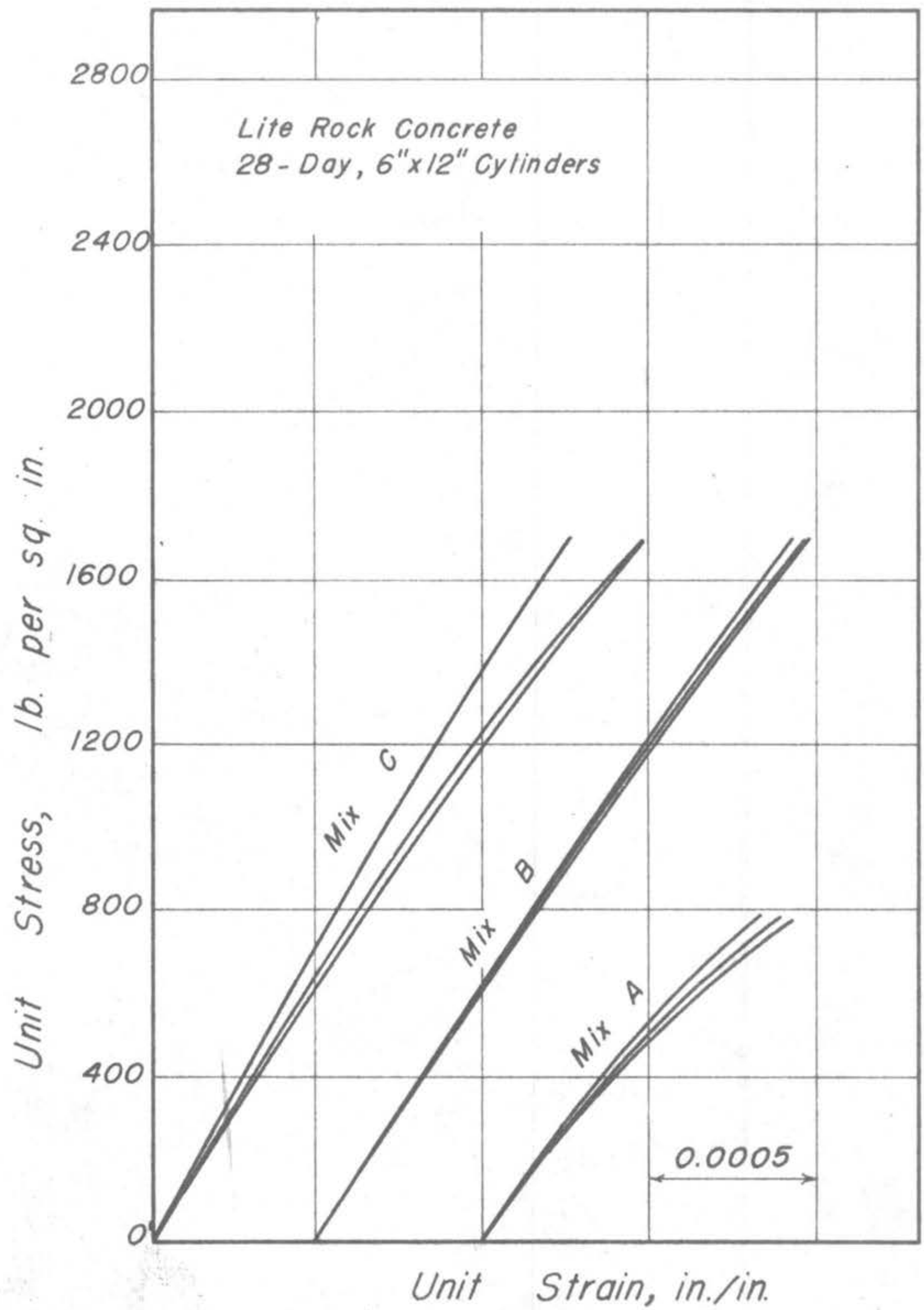

Fig. 13.– STRESS-STRAIN CURVES

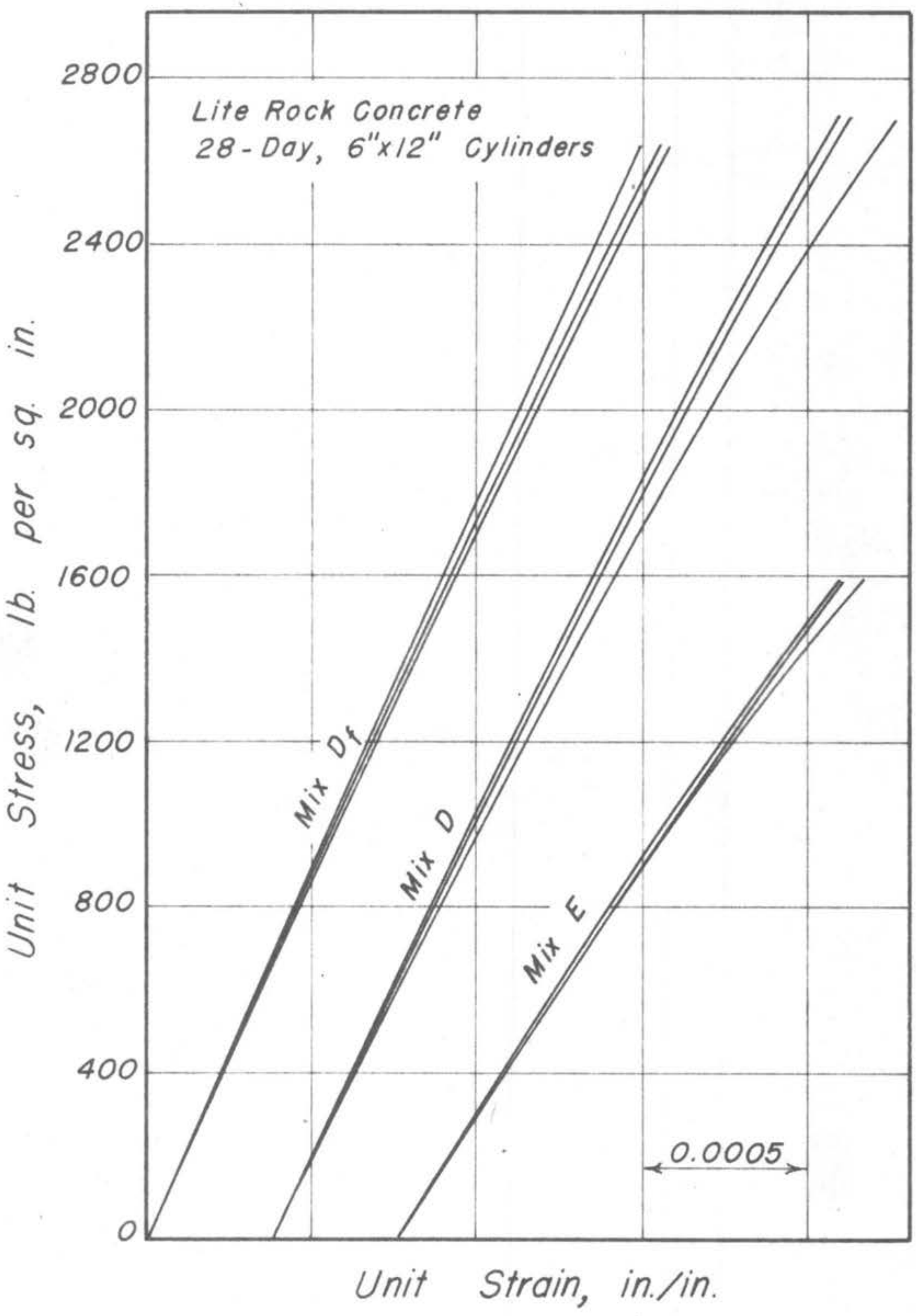

Fig. 14.– STRESS-STRAIN CURVES

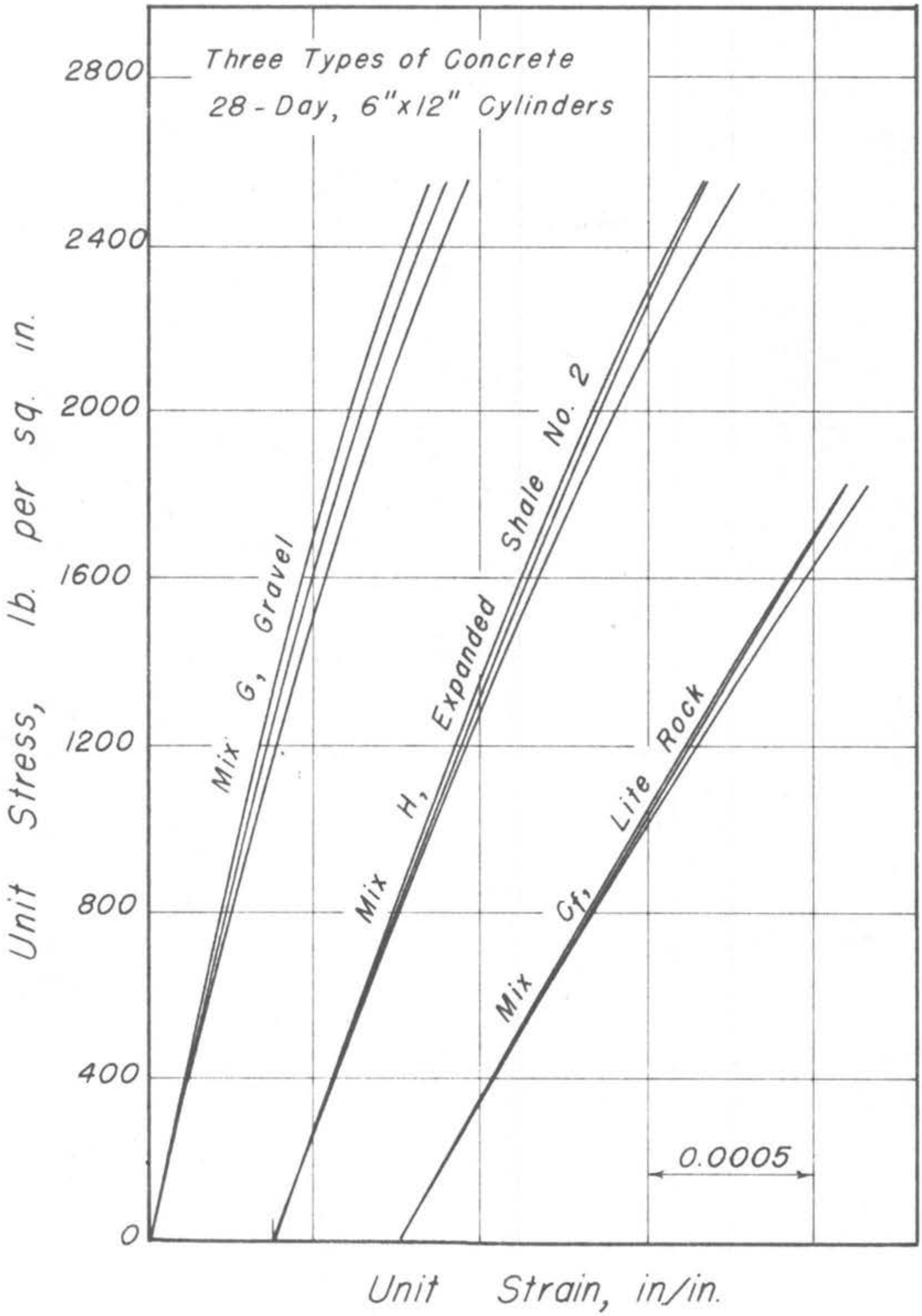

Fig. 15.– STRESS - STRAIN CURVES

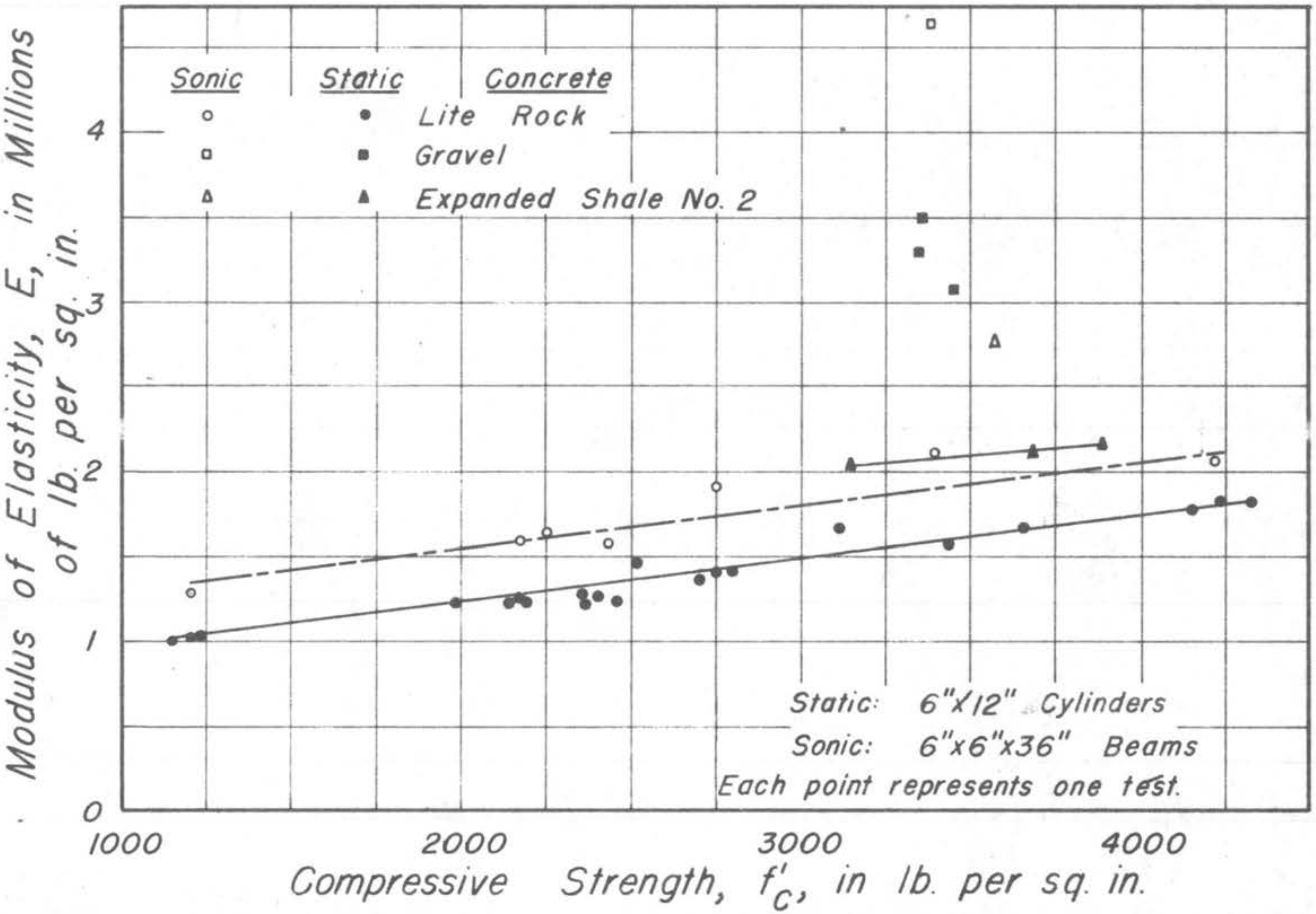

Fig. 16.— STATIC AND SONIC MODULUS OF ELASTICITY COMPARED TO STRENGTH FOR THREE TYPES OF CONCRETE

4. <u>Sonic modulus test</u> (<u>No. 6_s</u>). The test for flexure and the test for sonic modulus of elasticity were made on 6" x 6" x 36" plain-concrete beams cured moist. At twenty-eight days the specimens were removed from the fog room, weighed, and placed on the sonic modulus tester.

This apparatus, which is shown in Figure 17, 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 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 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.

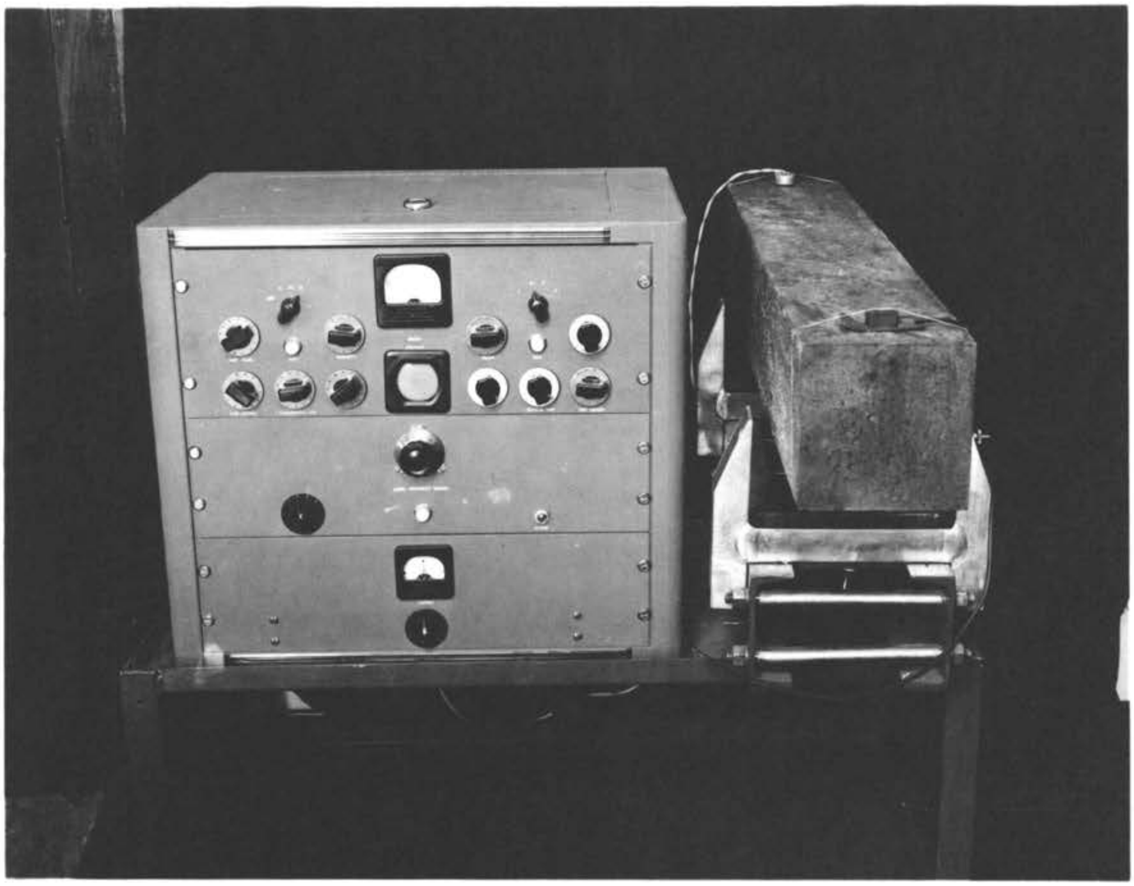

Figure 17. Sonic Modulus Test
on Plain Concrete Beam.

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

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 16 along with static modulus of elasticity.

5. Flexure test (No. 6). Immediately following the sonic modulus test, the specimen was removed and tested in flexure. An American beam tester, made by the American Beam Testing Company, was used. This device provides third point loading on an eighteen inch span and a gage which reads modulus of rupture for a 6" x 6" beam directly in pounds per square inch. The apparatus is pictured in Figure 18. Two breaks were made on each 36" beam and modulus of rupture was recorded to the nearest ten pounds per square inch. Average values for modulus of rupture are given in Table VI. They are plotted in Figure 19, and complete data are tabulated in the appendix.

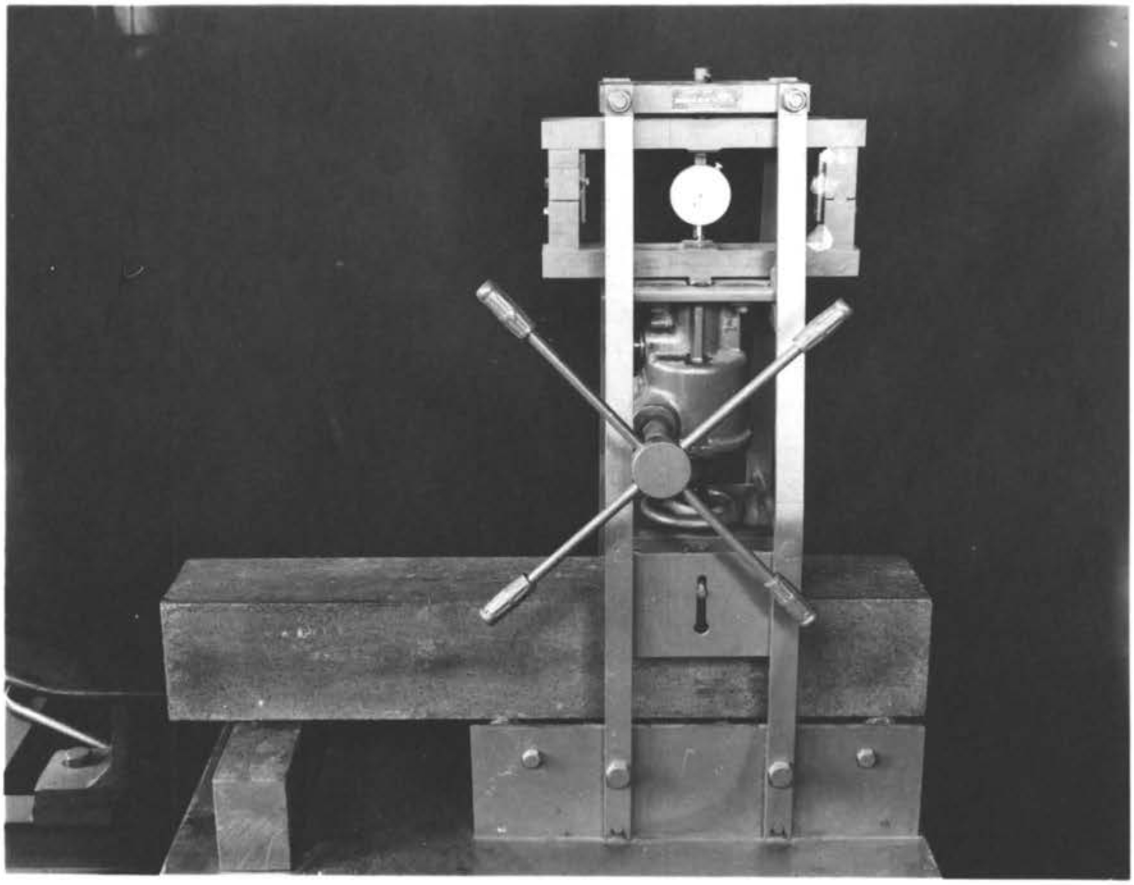

Figure 18. Flexure Test on Plain Concrete Beam.

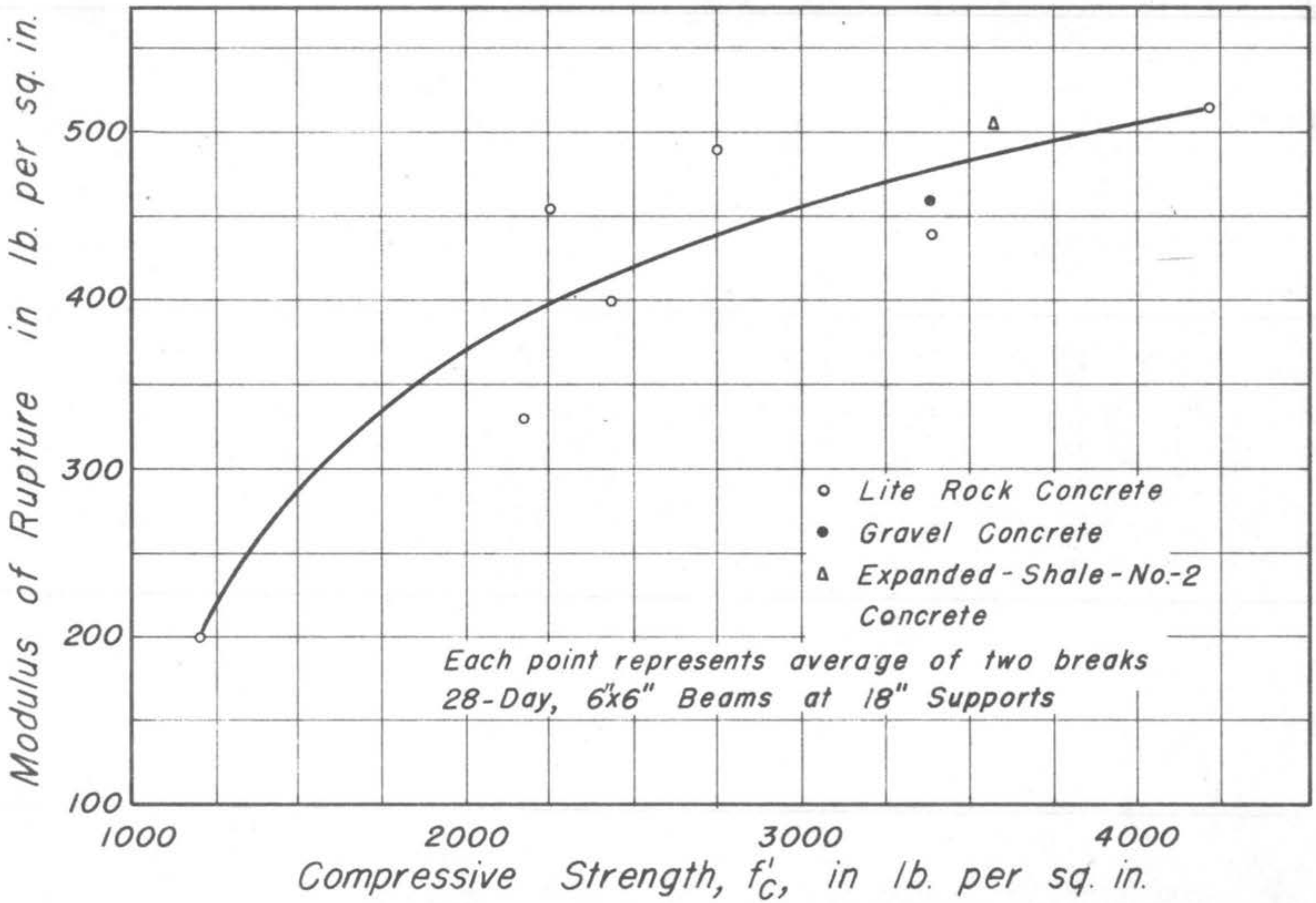

Fig. 19.– RELATION BETWEEN MODULUS OF RUPTURE AND COMPRESSIVE STRENGTH

TABLE VI

RESULTS OF FLEXURE TEST (NO. 6)

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

Each value is the average of two breaks.

6. Bond test (No. 7). Specimens for bond pull-out tests were 8" x 8" cylinders with 5/8" deformed bars extending about twenty inches below the bottom of the cylinder. The specimens were poured on a bench with holes provided for the reinforcing steel. They were cured seven days moist and twenty-one days in room air. To measure the initial end slip, a dial gage graduated to 0.0005 inches was used. A specimen ready for testing is shown in Figure 20.

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 inches per minute. Loads were recorded at end slip of 0.001 inches, and at the ultimate value. Results from the pull-out tests are shown in Table VII, plotted in Figure 21, and given in detail in the Appendix.

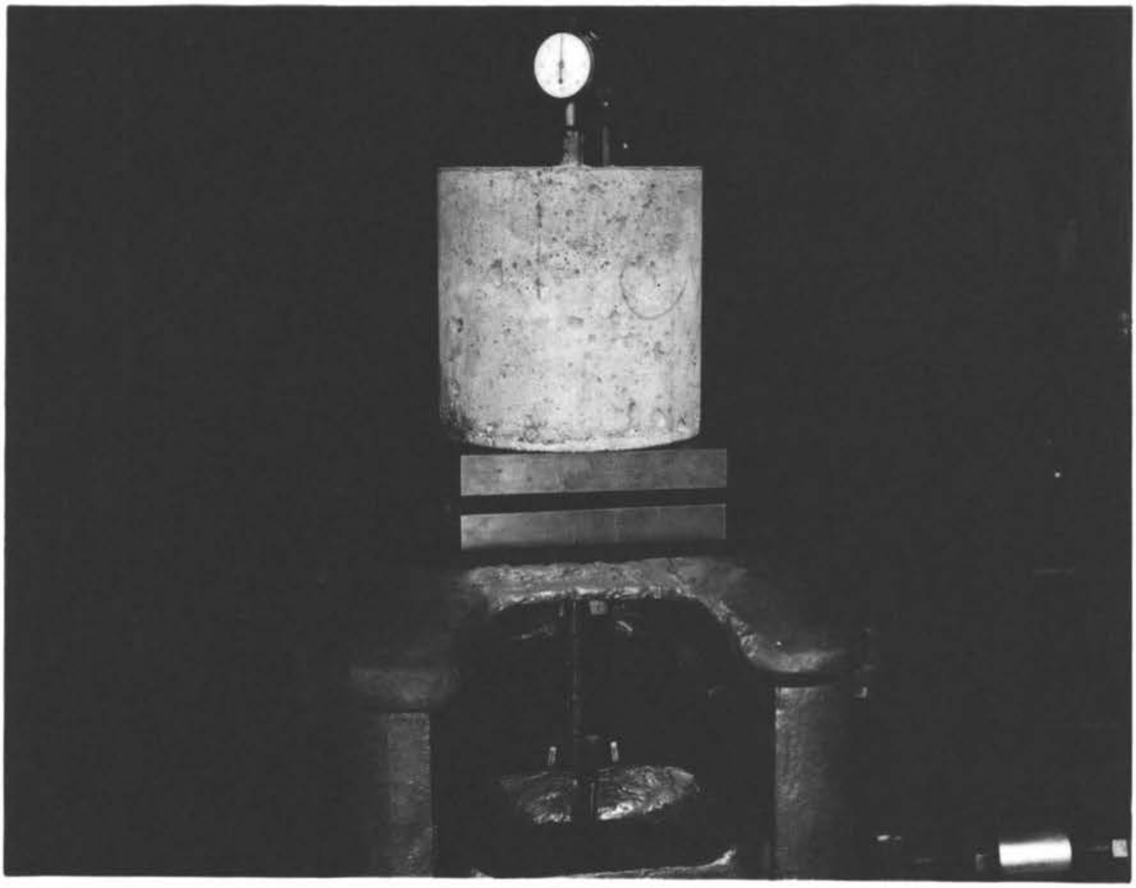

Figure 20. Bond Test on Pull-Out Specimen.

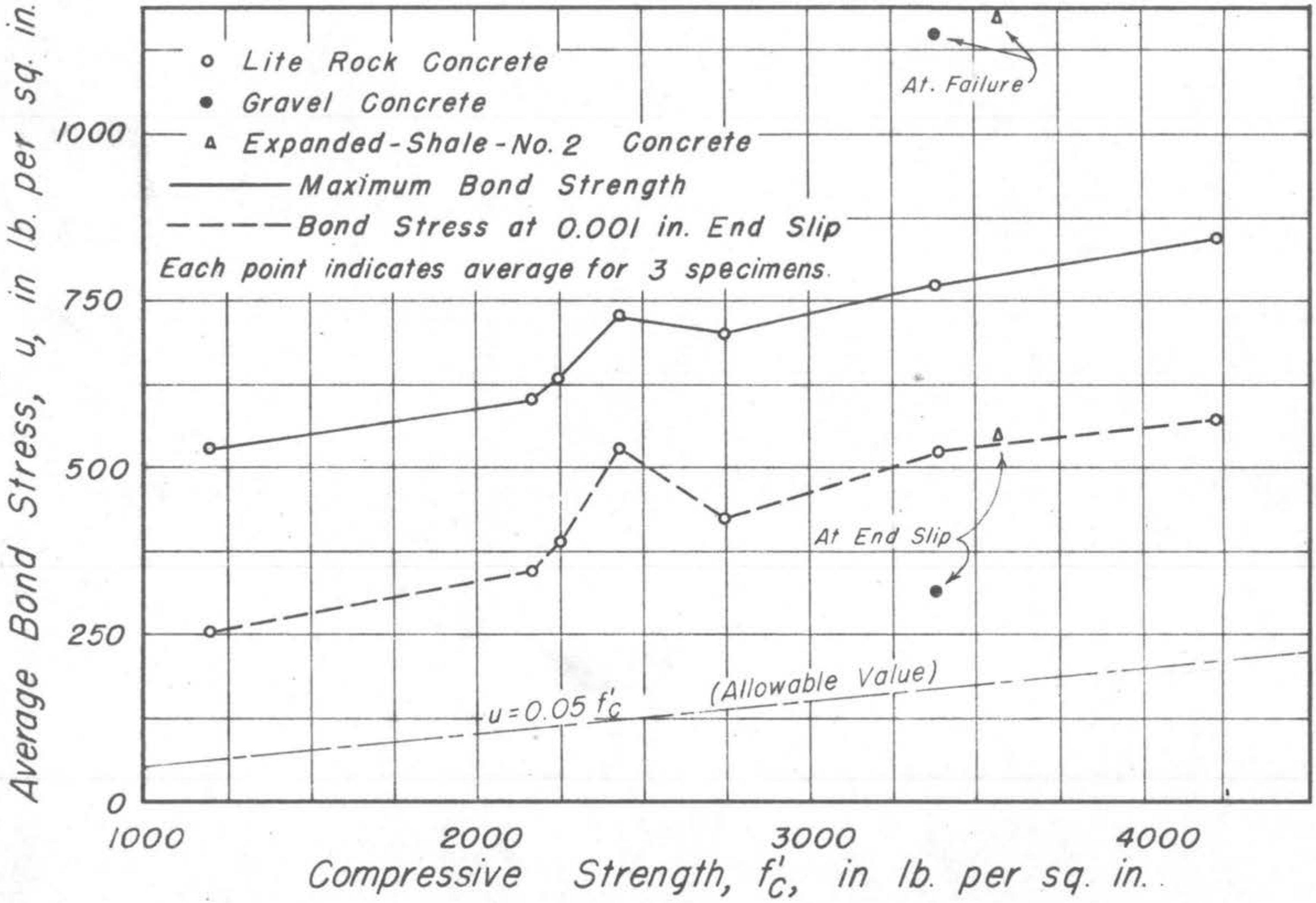

Fig. 21.- RELATION BETWEEN BOND AND COMPRESSIVE STRENGTH

TABLE VII

RESULTS OF BOND TEST

	Average Bond Stress, lb./sq. in.								
	Mix								
	A	B	C	D	C_f	D_f	E	G	H
At End Slip	256	351	528	523	426	572	392	317	549
At Failure	532	605	729	777	700	842	633	1163**	1178*

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

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. <u>Dorry abrasion test</u> (<u>No</u>. <u>8</u>). Specimens for the abrasion test were 2" x 4" cylinders cured seven days moist and twenty-one days in air. The abrasive material was crushed quartz between 30 and 40 mesh size. The abrasive was fed to a grinding disc which rotated approximately 30 times per minute. One thousand revolutions constituted a test.

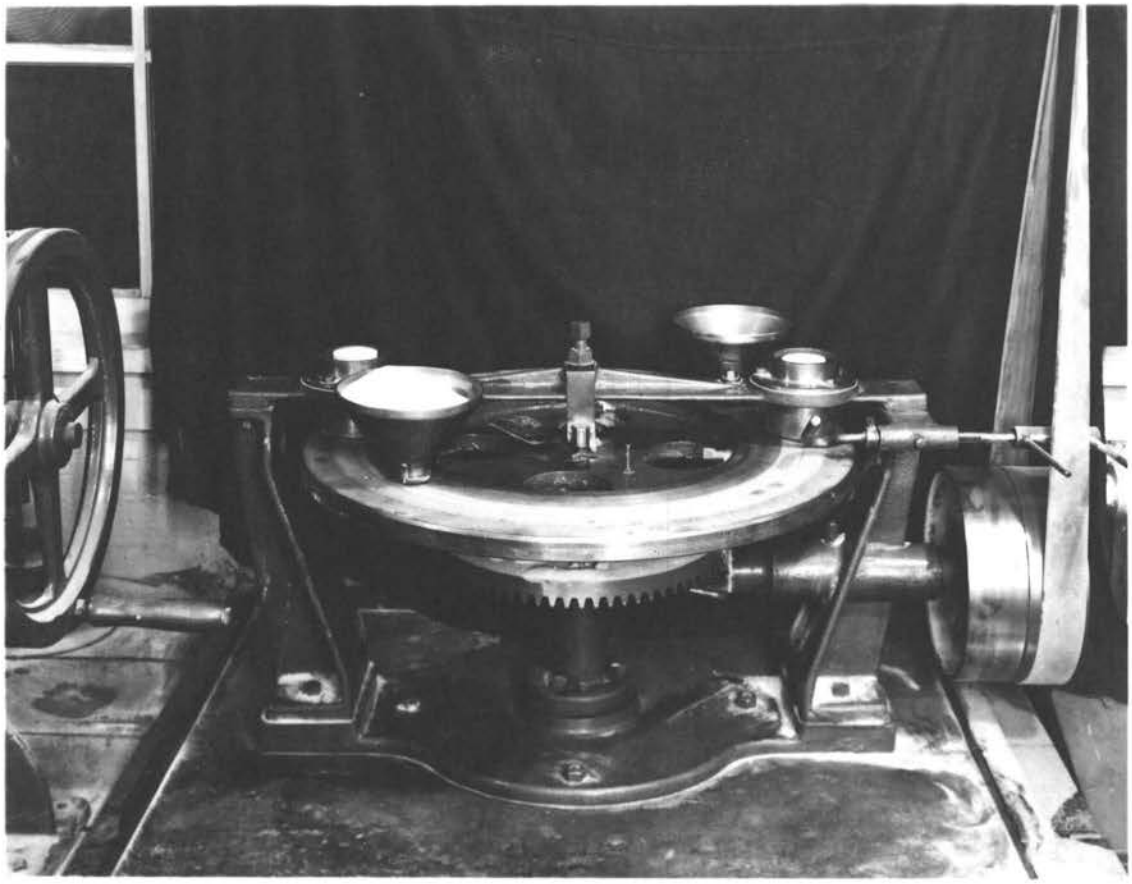

Figure 22. Dorry Abrasion Test
on Two Inch Cylinders.

The Dorry abrasion machine, which is pictured in Figure 22, 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 furnish a standard comparison.

Roughness was ground off the specimens before testing and they were then subjected to 1000 revolutions on the machine. They were next transferred to the opposite holder, turned end for end, and given a second 1000 revolutions. The average loss in grams for 1000 revolutions is recorded in Table VIII.

8. Absorption test (No. 9). Specimens for the absorption test were 4" x 8" cylinders, cured moist for seven days and in air for twenty-one 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 IX, and complete data are tabulated in the Appendix.

9. Shrinkage test (No. 10). Specimens for the shrinkage test were 3" x 3" x 11" bars into which 1/8" brass machine screws had been set for gage points at

TABLE VIII

RESULTS OF DORRY ABRASION TEST

Specimens: 2" x 4" cylinders   Curing: 7 days moist, 21 days air

Mix	Average Weight Loss in 1000 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

TABLE IX

SUMMARY OF ABSORPTION TEST RESULTS

Specimens: 4" x 8" cylinders   Curing: 7 days moist,
21 days air

24 Hour Absorption, per cent

	Mix						
	A	B	C_f	D_f	E	G	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.

Figure 23. Measurement of Shrinkage with Whittemore Strain Gage.

a ten-inch gage length. The brass screws had been drilled with a No. 60 drill as specified for the Whittemore strain gage which was used to measure shrinkage. The strain gage was calibrated to 0.0001 inch and was checked against a standard ten-inch invar bar. Readings could be repeated on this bar within 0.0001 inch. A measurement is illustrated in Figure 23.

Shrinkage specimens were measured at one day and at twenty-eight days. They were then oven dried, cooled, and measured again. Curing was seven days moist, and twenty-one 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 X.

TABLE X

SUMMARY OF SHRINKAGE TEST RESULTS

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

Condition	Shrinkage, per cent						
	Mix						
	A	B	C_f	D_f	E	G	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.

PART V-DISCUSSION

1. Mix design. The design of a Lite Rock concrete mixture differs from that for heavy concrete due to 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 seven-day compressive strength as a function of water-cement ratio. For the twenty-eight day curing period, however, the comparatively weak aggregate can not 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 2000-pound concrete with 3/4-inch aggregate, about 40 per cent of which is retained on a No. 4 sieve; 3000-pound concrete with 3/8-inch aggregate, about

35 per cent of which is retained on a No. 4 sieve; and 4000-pound concrete with 1/4-inch aggregate. Cement factors for these mixes should be about 5 1/2 sk./yd. for the first, and 9 sk./yd. for the second and third, these factors obtaining when approximately a three 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 weighs about one and one half times the weight of the coarse. Thus a proportion of coarse aggregate amounting to thirty per cent by weight is nearly forty per cent by volume.

The use of an air entraining or dispersing agent is not necessary 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 from the reduction in mixing water made possible, and the resulting increase in strength.

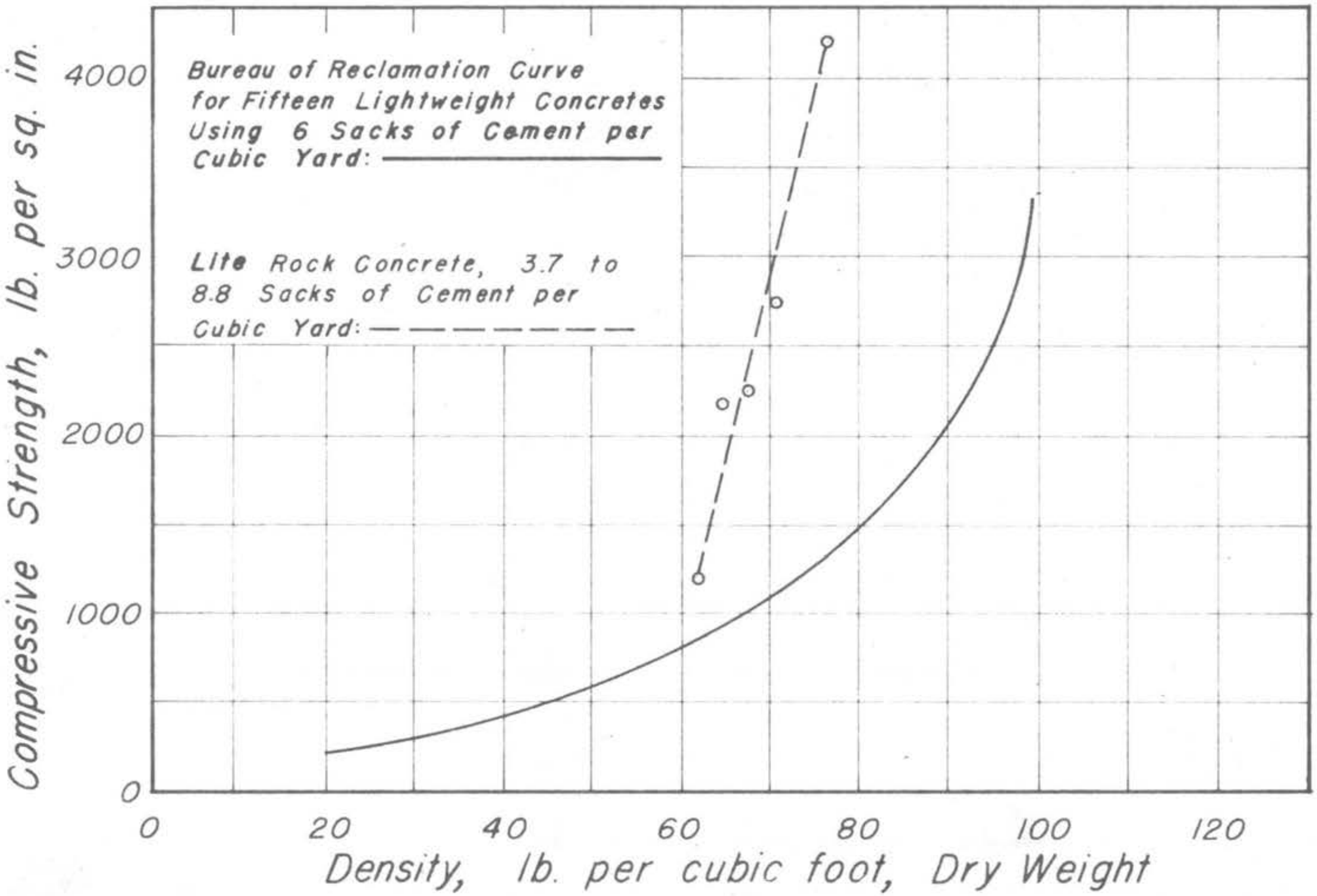

Fig. 24– STRENGTH of LIGHTWEIGHT CONCRETE AS a FUNCTION of DENSITY

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 lightweight concrete is limited by the degree of lightness. Lightweight concretes range from about thirty to one hundred and twenty-five 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. It does, however, make concrete stronger than any tested either by the Bureau of Reclamation or the National Bureau of Standards (1, pp.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 lightweight 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 1000 psi for concretes weighing less than 50 lb. per cu. ft. or above 2000 psi for concretes weighing less than 80 lb. per cu. ft., dry weight.

Lite Rock concrete is shown to be an exception to the above statement by Figure 24, where strengths of five Lite Rock concrete mixes are plotted against oven-dry weight. The Bureau of Reclamation curve in Figure 24 can not 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 yard is shown to advantage over the Bureau curve for six sack per yard concrete.

3. Effect of age on compressive strength. Due to 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 seven to twenty-eight days. Beyond twenty-eight 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 four-inch and six-inch cylinders. Results from the four-inch cylinders were not as consistent as desired. Flaws on the cylinder walls of four-inch cylinders have much larger effect and it is difficult to prevent eccentricity in loading. Results from the six-inch cylinders averaged about seven per cent higher than from the four inch with similar curing even though these six-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 16), showed good agreement with that for static modulus values. The modulus of elasticity of Lite Rock concrete may be stated very closely as follows:

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

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. Flexure strength. The flexure 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 19.

7. Bond strength. Very satisfactory results were obtained from the bond pull-out tests as is shown in Figure 21. At initial end slip of 0.001 in., both lightweight 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 VIII. The expanded-shale No. 2 concrete showed better resistance, but was still far under the gravel concrete. It is also necessary to point out that this comparison is by weight and that a volume comparison would show the lightweight concretes even less satisfactory for abrasive resistance.

9. Absorption. A comparison of absorption based on dry weight is unfair to any lightweight 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 only have 10 per cent by weight. Twenty-four hour absorption values for Lite Rock and the comparison shale concrete, shown in Table IX, 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.

PART VI-DESIGN OF LITE ROCK REINFORCED CONCRETE

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. Notation used here is explained on page .

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 lightweight 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 using $E = 1000\ f'_c$ for the gravel and

test values for the Lite Rock. The two moduli are plotted in Figure 25. The value of 1000 f'_c was used in accord with conventional 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 about three-eighths of the depth, d, below the surface of the 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 26 where moment factors for the two types of concrete are compared. From 25 to 35 per cent more moment is carried by the Lite Rock concrete than by the gravel concrete of equal compressive strength.

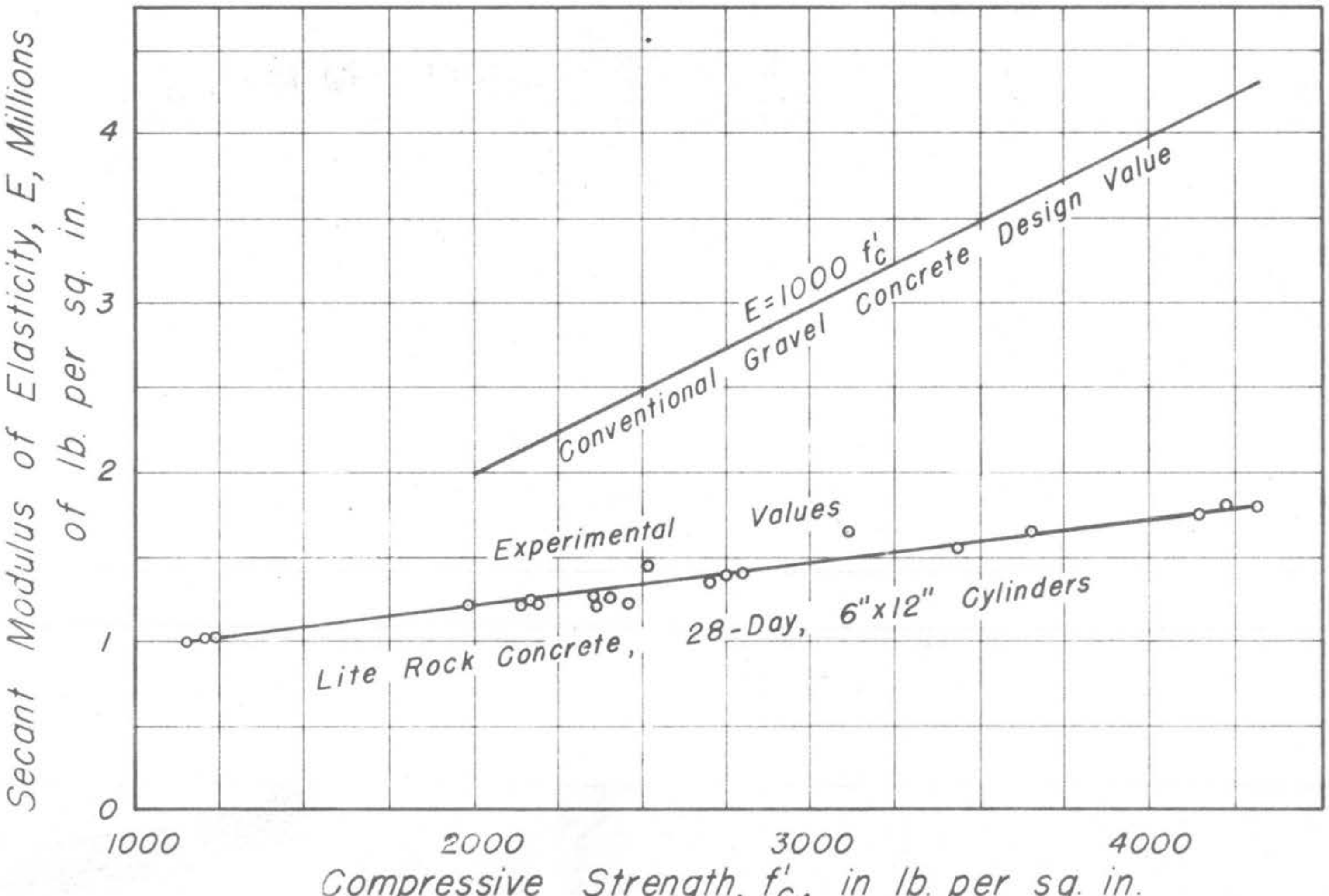

Fig. 25.– RELATION OF MODULUS OF ELASTICITY TO STRENGTH

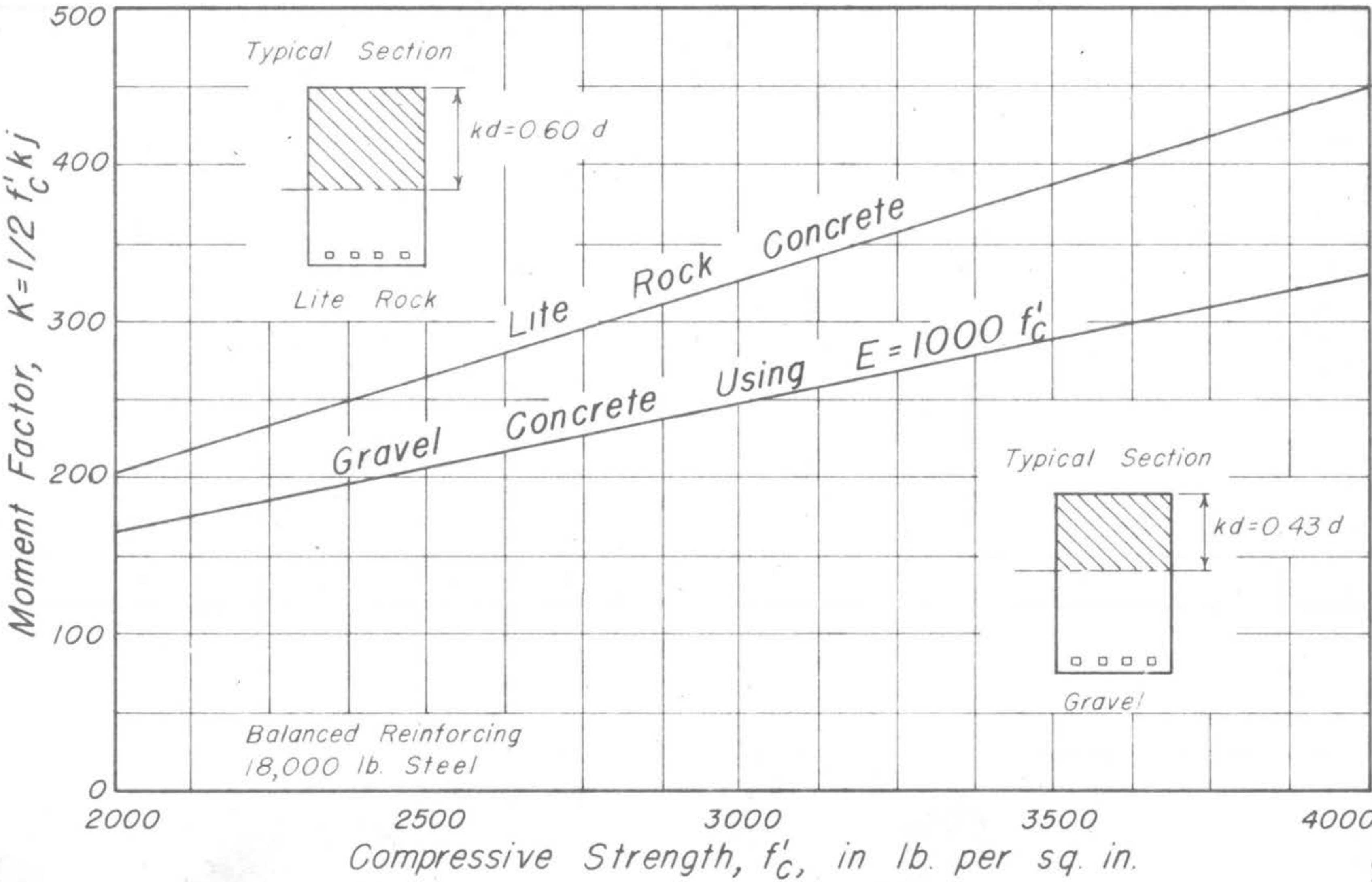

Fig. 26.– MOMENT FACTORS for TWO TYPES of CONCRETE

3. Design tables. Factors for the design of rectangular beams and slabs with Lite Rock concrete are given in Table XI. Factors for the review of beams are offered in Table XII.

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. However, 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 7/8" round bars for compression reinforcement. The stresses in the concretes are plotted against load in Figure 27. The value of the low modulus of elasticity with the consequent 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 and in a homogeneous beam deflection would increase as the value for E decreased.

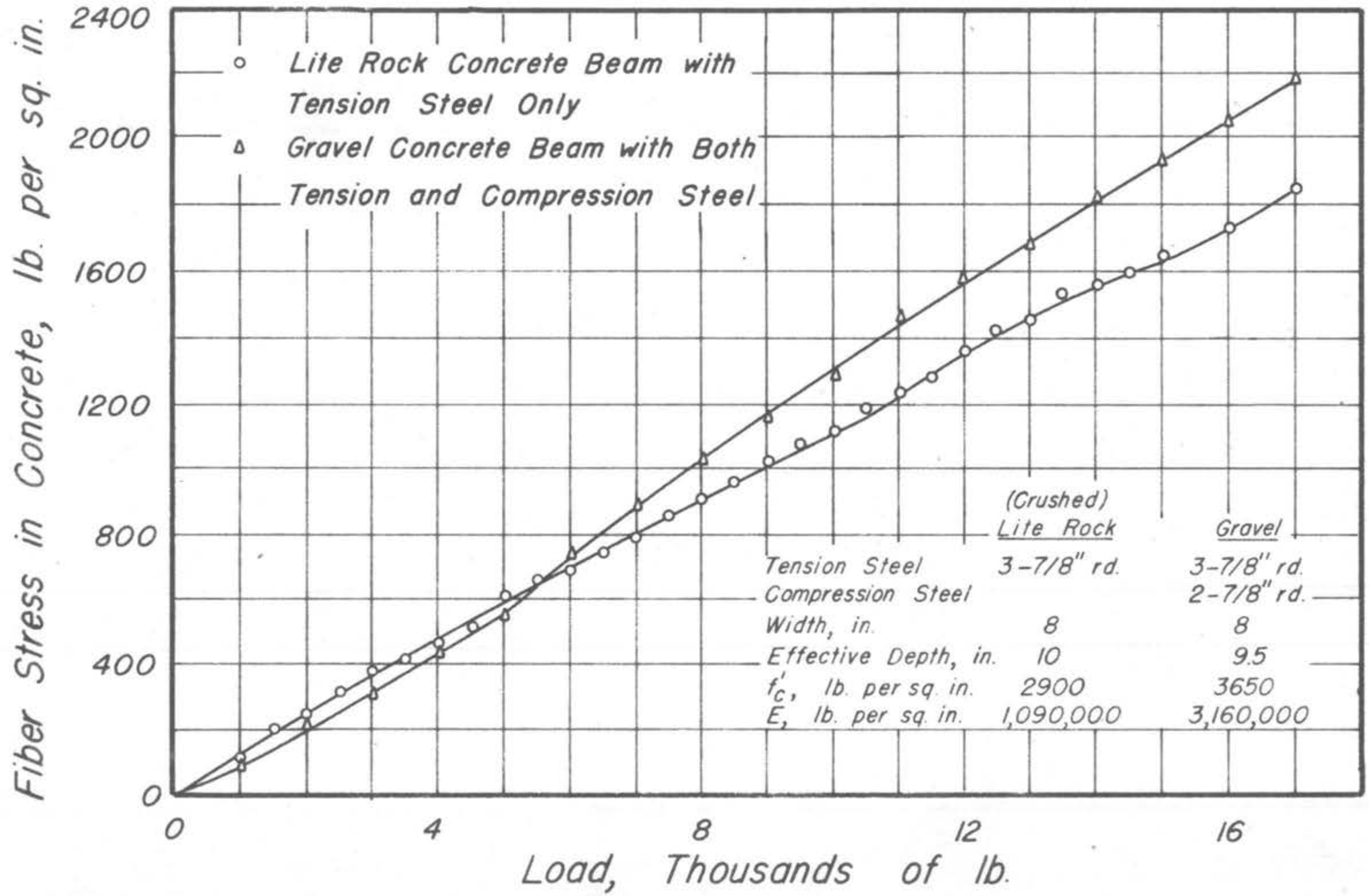

Fig. 27– COMPARISON BETWEEN REINFORCED CONCRETE BEAMS OF LITE ROCK AND GRAVEL CONCRETES

This might lead us to expect a doubly large deflection for Lite Rock-concrete members. However, an investigation at the University 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 the senior students more steel was used in the Lite Rock beams than in the gravel beams in proportion to the requirements for balanced reinforcing. Here the Lite Rock beams averaged five per cent more deflection than the gravel beams at a given load in the working range and fourteen 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, I, with the decreased modulus of elasticity, E, 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 I would give support to the experimental findings (5, p.76) while others would make I 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 allows less cracking below the neutral axis since tension stresses would only be half the value 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 bring 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 on 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.

TABLE XI
DESIGN OF LITE ROCK CONCRETE BEAMS AND SLABS

$k = \frac{n}{n+r}$ $\quad j = 1 - \frac{k}{3}$ $\quad p = \frac{n}{2r(n+r)}$ $\quad K = 1/2 f_c k j$ or $p f_s j$

(n) and f'_c	f_s	f_c	k	j	p	K
(18) 3750	18,000	1125	0.529	0.824	0.0165	245
		1500	0.600	0.800	0.0250	360
		1688	0.628	0.791	0.0295	420
	20,000	1125	0.503	0.832	0.0141	235
		1500	0.575	0.808	0.0216	349
		1688	0.603	0.799	0.0254	406
(20) 3000	18,000	900	0.500	0.833	0.0125	187
		1200	0.571	0.810	0.0190	277
		1350	0.600	0.800	0.0225	324
	20,000	900	0.474	0.842	0.0107	180
		1200	0.545	0.818	0.0164	268
		1350	0.575	0.808	0.0194	314
(22) 2500	18,000	750	0.478	0.841	0.0100	151
		1000	0.550	0.817	0.0153	225
		1125	0.579	0.807	0.0181	263
	20,000	750	0.452	0.849	0.0085	144
		1000	0.524	0.825	0.0131	216
		1125	0.553	0.816	0.0156	254
(24) 2000	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

TABLE XII
REVIEW OF LITE ROCK CONCRETE BEAMS AND SLABS

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

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	0.940	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	0.780	0.674	0.775
0.030	0.631	0.790	0.649	0.784	0.665	0.778	0.679	0.774

PART VII-CONCLUSIONS

The following conclusions are 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 1200 to 4200 pounds per square inch depending upon the cement factor and the maximum size aggregate.
5. Less strength is gained beyond the seven 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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APPENDIX

MIX DATA

	Mix		
	A	B	C
Date Poured	3-25-50	3-23-50	3-18-50
Proportions:			
Cement, lb.	11.79	17.25	25.00
Fine aggregate, lb.	36.00	31.50	32.90
Coarse aggregate, lb.	9.00	13.50	8.40
Dispersing agent, lb.	0.06	.10	.13
Water, lb.	12.62	11.70	12.13
Total batch weight, lb.	69.47	74.05	78.56
Approximate mixing time	8 min.	5 min.	5 min.
Average slump, in.	0.3	2.3	3.0
Workability	Good	Very good	Very good
Bleeding	Yes	No	No
Segregation	No	No	No
Fresh wt., 0.2 cu. ft.	15.29	15.97	15.03
Unit wt., lb./cu. ft.	76.45	79.85	75.15
Cement factor, sk./cu. yd.	3.7	5.4	6.9
Moisture content, % dry wt.			
Fine aggregate	6.6	6.6	2.4
Coarse aggregate	0.3	0.3	0.2
Water-cement ratio by wt.	1.07	0.68	0.49

MIX DATA

	Mix		
	D	C_f	D_f
Date Poured	3-15-50	5-20-50	5-13-50
Proportions:			
Cement, lb.	36.84	22.30	30.00
Fine aggregate, lb.	43.50	40.10	40.10
Coarse aggregate, lb.	2.72	---	---
Dispersing agent, lb.	0.16	0.13	0.16
Water, lb.	14.86	14.20	14.00
Total batch weight, lb.	98.08	76.73	84.26
Approximate mixing time	5 min.	5 min.	5 min.
Average slump, in.	5.0	4.6	5.3
Workability	Very good	Very good	Very good
Bleeding	No	No	No
Segregation	No	No	No
Fresh wt., 0.2 cu. ft., lb.	16.80	16.59	17.30
Unit wt., lb./cu. ft.	84.78	82.95	86.50
Cement factor, sk./cu. yd.	9.2	6.9	8.8
Moisture content, % dry wt.			
Fine aggregate	2.2*	0.0	0.0
Coarse aggregate		0.0	0.0
Water-cement ratio by wt.	0.40	0.64	0.47

*Combined

MIX DATA

	Mix		
	E	G	H
Date Poured	3-28-50	3-22-50	3-21-50
Proportions:			
Cement, lb.	25.40	25.20	23.40
Fine aggregate, lb.	33.00	78.60	44.40
Coarse aggregate, lb.	8.25	97.20	15.96
Dispersing agent, lb.	---	0.13	0.12
Water, lb.	15.40	15.28	12.76
Total batch weight, lb.	82.05	216.40	96.64
Approximate mixing time	5 min.	10 min.	5 min.
Average slump, in.	1.8	5.3	2.1
Workability	Very good	Good	Very good
Bleeding	No	No	No
Segregation	No	No	No
Fresh wt., 0.2 cu. ft., lb.	16.06	28.75	19.98
Unit wt., lb./cu. ft.	80.30	143.75	99.90
Cement factor, sk./cu. yd.	7.1	4.8	6.9
Moisture content, % dry wt.			
Fine aggregate	2.0	1.5	0.1
Coarse aggregate	0.0	1.1	0.0
Water-cement ratio by wt.	0.61	0.61	0.55

DATA ON SEVEN DAY COMPRESSIVE STRENGTH TEST
TEST NO. 1

Specimen: 4" x 8" cylinders   Curing: 7 day moist

Mix and Specimen No.		Date Tested	Dimensions, in.		Weight, lb.	Ultimate Load, lb.	Type of Break	Per cent Broken Aggre-gate	f'c, lb. per sq. in.
			Diam.	Height					
A	1	4-1-50	4.00	8.04	4.37	10690	Diag.	25	850
	2		4.00	8.00	4.37	9360	Cone	10	750
	3		4.00	7.92	4.27	9360	Cone	10	750
B	1	3-30-50	3.98	8.12	4.65	19200	Cone	60	1540
	2		3.97	7.92	4.56	20190	Diag.	70	1630
	3		4.00	8.05	4.85	23240	Diag.	60	1850
C	1	3-25-50	3.96	8.10	4.54	26870	Cone	75	2180
	2		3.97	8.06	4.52	29020	Diag.	75	2340
	3		4.03	8.02	4.56	29410	Cone	75	2300
D	1	3-24-50	3.99	8.15	4.82	34760	Diag.	75	2780
	2		4.00	8.10	4.74	34090	Diag.	75	2710
	3		4.00	8.14	4.82	37080	Diag.	75	2950
C_f	1	5-27-50	3.98	8.08	4.77	27350	Diag.	50	2200
	2		4.00	8.06	4.78	25990	Diag.	50	2070
	3		3.97	8.06	4.78	27940	Cone	50	2260
D_f	1	5-20-50	4.01	8.06	5.03	43050			3410
	2		3.99	8.16	5.02	44170			3530
	3		3.99	8.08	4.93	40240			3220

DATA ON SEVEN DAY COMPRESSIVE STRENGTH TEST
TEST NO. 1 (Cont'd.)

Specimen: 4" x 8" cylinders — Curing: 7 day moist

Mix and Specimen No.		Date Tested	Dimensions, in.		Weight, lb.	Ultimate Load, lb.	Type of Break	Per cent Broken Aggregate	f'c, lb. per sq. in.
			Diam.	Height					
E	1	4-4-50	3.98	8.06	4.73	26140	Diag.	60	2100
	2		3.97	8.05	4.53	22280	Cone	60	1800
	3		4.01	8.05	4.72	25840	Diag.	40	2050
G	1	3-29-50	3.98	8.08	8.24	25670	Cone		2060
	2		4.00	8.04	8.23	25700	Cone		2040
	3		4.01	8.02	8.30	25010	Cone		1980
H	1	3-28-50	4.00	8.08	5.83	21160	Cone	25	1680
	2		3.98	8.02	5.75	23030	Diag.	20	1850
	3		3.99	7.94	5.76	24340	Diag.	20	1950

DATA ON TWENTY-EIGHT DAY COMPRESSIVE STRENGTH TEST
TEST NO. 2

Specimen: 4" x 8" cylinders

Curing: 7 days moist,
21 days air

Mix and Specimen No.		Date Tested	Dimensions, in.		28 Day Weight, lb.	Ultimate Load, lb.	Type of Break	Per cent Broken Aggregate	f'c, lb. per sq. in.
			Diam.	Height					
A	1	4-22-50	4.00	8.04	3.94	13450	Cone	10	1070
	2		4.06	8.06	4.11	16910	Diag.	50	1310
	3		3.96	8.06	3.98	14980	Diag.	40	1220
B	1	4-20-50	3.98	8.00	4.19	25490	Diag.	70	2050
	2		3.98	8.04	4.33	24990	Diag.	70	2010
	3		3.96	8.02	4.23	25860	Cone	70	2100
C	1	4-15-50	3.98	8.04	4.28	32570	Diag.	70	2620
	2		3.97	8.06	4.31	33400	Diag.	70	2700
	3		3.97	8.05	4.34	33480	Cone	70	2700
D	1	4-14-50	3.99	8.05	4.61	36790	Cone	90	2940
	2		3.99	7.94	4.56	36990	Diag.	90	2960
	3		3.99	8.08	4.62	33650	Diag.	90	2690
C_f	1	6-17-50	4.00	8.00	4.45	36660	Diag.	50	2920
	2		4.02	8.04	4.45	35900	Diag.	50	2830
	3		3.99	8.02	4.33	36070	Cone	50	2890
D_f	1	6-10-50	4.02	8.06	4.81	34420	Diag.	50	2710
	2		4.05	8.02	4.95	47230	Diag.	50	3670
	3		4.00	8.10	4.87	51180	Diag.	50	4070

DATA ON TWENTY-EIGHT DAY COMPRESSIVE STRENGTH TEST
TEST NO. 2 (Cont'd.)

Specimen: 4" x 8" cylinders

Curing: 7 days moist, 21 days air

Mix and Specimen No.		Date Tested	Dimensions, in.		28 Day Weight, lb.	Ultimate Load, lb.	Type of Break	Per cent Broken Aggregate	f'c, lb. per sq. in.
			Diam.	Height					
E	1	4-25-50	3.96	8.04	4.59-	33370	Cone	70	2710
	2		4.02	8.08	4.54	30070	Cone	70	2370
	3		3.97	8.07	4.49	29970	Diag.	50	2420
G	1	4-19-50	3.98	7.96	7.71	38050	Cone		3060
	2		4.05	8.06	8.16	40800	Cone		3170
	3		3.97	8.02	7.79	37920	Diag.		3060
H	1	4-18-50	4.00	8.03	5.59	38220	Diag.	40	3040
	2		3.98	8.04	5.72	38790	Diag.	40	3120
	3		3.97	8.10	5.66	38070	Diag.	40	3080

DATA ON TWENTY-EIGHT DAY COMPRESSIVE STRENGTH TEST
TEST NO. 3

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

Mix and Specimen No.	Date Tested	Dimensions, in.		28 Day Weight, lb.	Ultimate Load, lb.	Type of Break	Per cent Broken Aggregate	f'c, lb. per sq. in.
		Diam.	Height					
A 1	4-22-50	4.00	8.04	4.44	11420	Diag.	15	910
2		3.94	8.06	4.37	13480	Diag.	25	1110
3		4.00	8.06	4.47	14660	Diag.	35	1170
B 1	4-20-50	3.98	8.02	4.80	24630	Diag.	70	1990
2		3.97	7.92	4.65	24130	Cone	70	1950
3		3.97	8.12	4.71	24130	Diag.	70	1950
C 1	4-15-50	3.98	8.10	4.53	31990	Diag.	50	2570
2		3.98	8.12	4.54	32030	Diag.	70	2570
3		3.94	8.06	4.49	32020	Diag.	60	2630
D 1	4-14-50	4.00	8.08	4.90	37840	Diag.	90	3010
2		3.97	8.08	4.84	33060	Diag.	75	2670
3		3.97	8.08	4.88	37040	Diag.	90	2990
C_f 1	6-17-50	4.01	8.12	4.77	32270	Cone	50	2560
2		3.97	8.02	4.64	33840	Diag.	50	2730
3		4.00	8.06	4.74	36100	Diag.	50	2870
D_f 1	6-10-50	3.97	8.04	4.92	48210	Cone	50	3890
2		3.98	8.02	4.85	48960	Cone	50	3940
3		3.98	8.02	4.92	43450	Cone	50	3490

DATA ON TWENTY-EIGHT DAY COMPRESSIVE STRENGTH TEST
TEST NO. 3 (Cont'd.)

Specimen: 4" x 8" cylinders Curing: 28 days moist

Mix and Specimen No.		Date Tested	Dimensions, in.		28 Day Weight, lb.	Ultimate Load, lb.	Type of Break	Per cent Broken Aggregate	f'c, lb. per sq. in.
			Diam.	Height					
E	1	4-25-50	3.97	8.02	4.86	33720	Cone	70	2720
	2		4.00	8.06	4.72	29880	Cone	70	2380
	3		3.97	8.02	4.67	29000	Diag.	50	2340
G	1	4-19-50	3.99	8.06	8.26	40220	Cone		3220
	2		4.01	8.10	8.25	39810	Cone		3150
	3		3.98	8.08	8.28	38580	Diag.		3100
H	1	4-18-50	4.00	8.08	5.91	34860	Diag.	40	2770
	2		3.98	8.00	5.87	37660	Diag.	40	3030
	3		3.99	8.00	5.86	36170	Diag.	40	2890

RESULTS OF TEST No. 4
COMPRESSION

General Data:

Apparatus: Graf strainometer used as a compressometer.

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

Loading: Increments of 2000 or 3000 lb. at maximum speed of 0.055 in. per min.

Typical Calculations:

LOAD, lb.	$\frac{\text{Load}}{\text{Area}}$ UNIT STRESS, lb. per sq. in.	GAGE READING 0.001 in.	CORRECTED GAGE READING 0.001 in.	$\frac{\text{Gage Reading}}{2}$ DEFORMATION 0.001 in.	$\frac{\text{Deform.}}{10}$ UNIT STRAIN 0.001 in. per in.
2000	71	1.0	1.2	.6	.06
4000	143	2.2	2.4	1.2	.12
6000	214	3.5	3.7	1.95	.195
8000	285	4.8	5.0	2.5	.25
10000	356	6.1	6.3	3.15	.315
12000	427	7.6	7.8	3.9	.39
14000	498	9.2	9.4	4.7	.47
16000	568	10.6	10.8	5.4	.54
18000	639	12.5	12.7	6.35	.635
20000	710	14.5	14.7	7.35	.735
22000	781	16.4	16.6	8.3	.83
34110	$1210 = f'_c$	Failure			

0.45 (ultimate load) = 15580 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 NO. A-4

Date: 4-22-50

Load, lb.	Cylinder No. and Dimensions					
	1 5.99" x 11.94"		2 5.99" x 12.02"		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.
2000	71	6.0	71	6.0	71	7.0
4000	143	12.0	143	13.5	142	13.0
6000	214	19.5	214	21.0	214	19.0
8000	285	25.0	285	28.0	285	26.5
10000	356	31.5	356	35.5	356	33.0
12000	427	39.0	427	44.0	427	40.0
14000	498	47.0	498	52.0	498	48.0
16000	568	54.0	568	61.5	570	57.0
18000	639	63.5	639	71.5	641	66.0
20000	710	73.5	710	81.0	712	76.5
22000	781	83.0	781	91.5	783	87.5
32230					1147	Failure
34110	1210	Failure				
34620			1229	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 NO. B-4

Date: 4-20-50

Load, lb.	Cylinder No. and Dimensions					
	1 5.98" x 12.00"		2 5.97" x 12.10"		3 5.98" x 12.08"	
	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.
2000	71	6.0	71	5.0	71	5.5
4000	142	12.0	143	12.0	142	12.0
6000	214	18.0	214	18.0	214	17.5
8000	285	24.0	286	23.5	285	23.0
10000	356	29.0	357	29.5	356	28.5
12000	427	34.0	429	35.5	427	34.0
14000	498	40.0	500	41.0	498	40.0
16000	570	46.0	572	47.0	570	45.5
18000	641	51.5	643	52.5	641	51.0
20000	712	57.0	715	59.5	712	57.0
22000	783	63.0	786	65.0	783	63.0
24000	854	68.5	857	71.0	854	70.0
26000	926	74.5	929	77.0	926	75.5
28000	997	81.0	1000	82.5	997	82.0
30000	1068	86.0	1072	88.5	1068	87.5
32000	1139	92.0	1143	95.0	1139	94.0
34000	1210	97.5	1215	101.5	1210	100.5
36000	1282	104.0	1286	107.0	1282	107.0
38000	1353	110.5	1358	114.0	1353	114.0
40000	1424	117.5	1429	120.0	1424	120.5
42000	1495	123.0	1500	127.0	1495	128.0
44000	1566	130.0	1572	134.0	1566	134.5
46000	1638	136.5	1643	140.0	1638	142.0
48000	1709	143.0	1715	149.0	1709	150.5
59840			2138	Failure		
60950	2170	Failure				
61590					2193	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 NO. C-4

Date: 4-15-50

Load, lb.	Cylinder No. and Dimensions					
	1 5.99" x 12.02"		2 5.98" x 12.04"		3 5.99" 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.
3000	106		107	8.0	106	8.0
6000	213	14.5	213	17.5	213	16.5
9000	319	21.0	320	23.5	319	
12000	426	28.5	427	32.0	426	34.0
15000	532	35.5	534		532	
18000	639	44.5	640	51.0	639	52.0
21000	745	51.0	748		745	61.0
24000	852	58.5	854	67.0	852	70.0
27000	958	65.5	961	75.0	958	79.0
30000	1065	72.5	1068	84.0	1065	87.5
33000	1171	81.5	1175	94.0	1171	97.0
36000	1278	90.0	1281	103.0	1278	105.0
39000	1384	98.0	1388	113.5	1384	115.0
42000	1490	106.5	1495	124.0	1490	126.0
45000	1597	115.0	1602	136.0	1597	136.5
48000	1703	125.5	1709	145.0	1703	147.0
51000	1810	134.5	1816	157.0	1810	157.5
54000	1916	144.5	1922	167.5	1916	172.0
57000	2022	153.0	2029	177.5	2022	186.0
60000	2129	164.5	2136	193.5	2129	199.0
66460					2358	Failure
67440			2401	Failure		
71000	2520	Failure				
0.45 Max.	1134	78.0	1080	85.0	1061	87.5
E, lb./in.^{2}	1.45×10^6		1.27×10^6		1.21×10^6	

DATA ON COMPRESSION TEST
TEST NO. D-4

Date: 4-14-50

Load, lb.	Cylinder No. and Dimensions					
	1 6.01" x 12.00"		2 5.98" x 12.00"		3 6.00" 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.
3000	106	6.5	107	6.0	106	
6000	212	13.0	214	12.5	212	
9000	317	13.0	320	18.5	318	
12000	423	25.5	427	25.0	424	
15000	529	34.0	534	32.0	531	
18000	635	40.5	641	38.0	637	
21000	740	48.0	749	44.0	743	
24000	846	54.5	854	51.0	849	49.0
27000	951	61.5	961	58.0	955	56.0
30000	1057	68.5	1068	64.5	1061	62.5
33000	1163	75.5	1175	71.5	1167	68.5
36000	1269	83.0	1282	78.0	1273	75.5
39000	1375	90.5	1388	84.5	1380	82.5
42000	1481	96.5	1495	91.0	1486	79.0
45000	1586	103.0	1602	98.5	1592	96.0
48000	1692		1709	105.0	1698	102.5
51000	1798	117.5	1816	111.5	1804	109.0
54000	1904	126.5	1922	118.0	1910	116.0
57000	2009	132.5	2029	125.0	2016	123.0
60000	2115	140.5	2136	132.0	2122	130.5
63000	2221	148.5	2243	138.5	2229	137.0
66000	2327	156.5	2350	146.0	2335	144.0
69000	2432	164.0	2456	153.0	2441	152.0
72000	2538	172.5	2563	161.0	2547	159.0
75000	2644	182.5	2670	168.0	2653	166.0
78000	2750	193.0	2777	176.0	2759	173.5
81000	2855	202.0	2884	184.0	2865	182.0
87340			3109	Failure		
97100	3423	Failure				
103040					3645	Failure
0.45 Max.	1540	99.5	1399	85.0	1640	99.0
E, lb./in.^{2}	1.55×10^6		1.65×10^6		1.66×10^6	

DATA ON COMPRESSION TEST
TEST NO. C_f-4

Date: 6-17-50

Load, lb.	Cylinder No. and Dimensions					
	1 5.99" x 12.00"		2 5.98" x 11.96"		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.
3000	106	8.5	107	11.0	107	6.5
6000	213	16.0	213	20.5	213	14.0
9000	319	24.0	320	28.0	320	22.0
12000	426	31.5	427	31.5	427	29.0
15000	532	39.0	534	39.0	534	37.0
18000	639	47.0	640	46.0	640	43.5
21000	745	54.5	748	53.0	748	51.5
24000	852	62.0	854	61.0	854	60.5
27000	958	70.5	961	68.5	961	68.0
30000	1065	78.5	1068	76.5	1068	76.0
33000	1171	86.5	1175	83.5	1175	84.0
36000	1278	94.5	1281	91.5	1281	92.0
39000	1384	103.0	1388	99.0	1388	99.5
42000	1490	112.0	1495	107.0	1495	108.0
45000	1597	121.0	1602	116.0	1602	116.5
48000	1703	129.5	1709	125.5	1709	126.0
51000	1810	139.0	1816	133.5	1816	136.0
54000	1916	148.5	1922	143.0	1922	144.0
76100	2700	Failure				
77160			2747	Failure		
78500					2795	Failure
0.45 Max.	1215	89.5	1236	88.5	1258	90.0
E, lb./in.^{2}	1.36×10^6		1.40×10^6		1.40×10^6	

DATA ON COMPRESSION TEST
TEST NO. D_f-4

Date: 6-10-50

Load, lb.	Cylinder No. and Dimensions					
	1 6.00" x 12.00"		2 5.98" x 11.96"		3 5.97" 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.
3000	106	6.5	107	6.0	107	6.5
6000	212	12.5	214	12.0	214	13.0
9000	318	17.5	320	18.0	322	19.0
12000	424	23.0	427	24.0	429	25.0
15000	531	29.0	534	30.0	536	31.5
18000	637	35.0	641	35.5	643	37.5
21000	743	41.0	748	41.5	750	43.0
24000	849	47.0	854	47.5	858	49.0
27000	955	53.0	961	54.0	965	56.0
30000	1061	58.5	1068	60.0	1072	62.0
33000	1167	64.5	1175	65.5	1179	68.5
36000	1273	70.0	1282	71.5	1286	74.5
39000	1380	76.0	1388	77.5	1394	81.0
42000	1486	82.0	1495	83.0	1501	86.5
45000	1592	87.5	1602	89.0	1608	92.5
48000	1698	94.5	1709	95.5	1715	98.0
51000	1804	98.5	1816	101.5	1822	104.0
54000	1910	105.5	1922	108.0	1930	111.5
57000	2016	111.5	2029	114.5	2037	117.5
60000	2122	118.0	2136	120.5	2144	124.0
63000	2229	124.0	2243	126.5	2251	131.0
66000	2335	130.0	2350	133.0	2358	137.5
69000	2441	136.0	2456	140.0	2466	143.5
72000	2547	143.0	2563	146.0	2573	150.5
75000	2653	150.0	2670	152.5	2680	157.5
78000	2759	156.5	2777	159.5	2787	164.0
81000	2865	163.0	2884	166.0	2894	172.5
84000	2971	170.0	2990	172.5	3002	179.0
115980					4144	Failure
119440	4225	Failure				
121020			4315	Failure		
0.45 Max.	1901	105.5	1942	108.5	1865	107.0
E, lb./in.^{2}	1.80×10^6		1.79×10^6		1.74×10^6	

DATA ON COMPRESSION TEST
TEST NO. E-4

Date: 4-25-50

Load, lb.	Cylinder No. and Dimensions					
	1 5.99" x 11.96"		2 5.99" x 11.97"		3 5.99" 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.
3000	106	8.0	106	9.0	106	8.5
6000	213	16.5	213	17.0	213	17.5
9000	319	25.5	319	26.0	319	25.5
12000	426	34.0	426	34.0	426	33.5
15000	532	43.0	532	42.5	532	41.5
18000	639	51.0	639	51.5	639	50.0
21000	745	60.0	745	60.0	745	59.0
24000	852	69.5	852	69.0	852	67.5
27000	958	78.0	958	77.5	958	76.5
30000	1065	88.0	1065	86.0	1065	85.5
33000	1171	98.0	1171	95.5	1171	94.0
36000	1278	107.0	1278	104.0	1278	102.5
39000	1384	117.0	1384	113.5	1384	112.5
42000	1490	128.0	1490	124.0	1490	122.0
45000	1597	138.5	1597	133.0	1597	131.0
56000	1987					
66260					2351	
68120			2417			
0.45 Max.	894	73.0	1088	88.5	1058	83.5
E, lb./in.^{2}	1.22×10^6		1.23×10^6		1.27×10^6	

DATA ON COMPRESSION TEST
TEST NO. G-4

Date: 4-19-50

Load, lb.	1 5.99" x 12.00" Unit Stress, lb. per sq. in.	1 5.99" x 12.00" Unit Strain, 10^{-5}in. per in.	2 5.98" x 12.08" Unit Stress, lb. per sq. in.	2 5.98" x 12.08" Unit Strain, 10^{-5}in. per in.	3 6.00" x 12.06" Unit Stress, lb. per sq. in.	3 6.00" x 12.06" Unit Strain, 10^{-5}in. per in.
3000	106	3.5	107	2.5	106	2.0
6000	213	7.0	214	6.0	212	5.0
9000	319	10.0	320	9.0	318	7.5
12000	426	13.5	427	12.5	424	11.0
15000	532	17.0	534	15.5	531	13.5
18000	639	20.0	641	18.5	637	16.5
21000	745	23.5	748	21.5	743	19.5
24000	852	27.0	854	25.0	849	22.5
27000	958	30.5	961	28.0	955	26.0
30000	1065	34.0	1068	31.5	1061	29.0
33000	1171	37.5	1175	34.5	1167	32.0
36000	1278	41.5	1282	38.0	1273	36.0
39000	1384	45.0	1388	41.5	1380	39.0
42000	1490	48.5	1495	44.0	1486	42.5
45000	1597	52.5	1602	48.0	1592	46.0
48000	1703	56.5	1709	51.5	1698	49.0
51000	1810	60.0	1816	55.5	1804	53.0
54000	1916	65.0	1922	58.5	1910	57.0
57000	2023	68.0	2029	62.5	2016	61.0
60000	2129	73.5	2136	67.5	2122	64.0
63000	2226	77.5	2243	72.0	2229	68.5
66000	2342	83.0	2350	77.0	2335	73.5
69000	2449	88.5	2456	82.0	2441	78.5
72000	2555	93.5	2563	87.0	2547	83.5
93890			3342	Failure		
94830					3354	Failure
97290	3452	Failure				
0.45 Max.	1553	50.5	1504	45.5	1509	43.0
E, lb./in.^{2}	3.08×10^6		3.31×10^6		3.51×10^6	

DATA ON COMPRESSION TEST
TEST NO. H-4

Date: 4-18-50

Load, lb.	Cylinder No. and Dimensions					
	1 5.99" x 12.04"		2 5.99" x 12.02"		3 6.00" 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.
3000	106	5.5	106	4.0	106	5.0
6000	213	10.0	213	9.0	212	10.5
9000	319	15.0	319	13.5	318	15.0
12000	426	20.0	426	19.0	424	20.0
15000	532	25.5	532	24.0	531	25.0
18000	639	30.5	639	29.0	637	30.0
21000	745	35.5	745	34.0	743	35.0
24000	852	41.0	852	38.0	849	39.5
27000	958	46.5	958	43.0	955	44.5
30000	1065	52.0	1065	48.0	1061	49.5
33000	1171	57.0	1171	52.5	1167	54.5
36000	1278	62.5	1278	57.5	1273	60.0
39000	1384	68.0	1384	62.5	1380	65.0
42000	1490	73.5	1490	68.0	1486	70.0
45000	1597	80.0	1597	73.5	1592	75.5
48000	1703	85.0	1703	79.0	1698	80.5
51000	1810	90.5	1810	84.5	1804	85.5
54000	1916	97.5	1916	90.5	1910	90.5
57000	2022	103.5	2022	95.5	2016	96.0
60000	2129	110.5	2129	102.0	2122	102.0
63000	2236	116.5	2236	108.0	2229	108.0
66000	2342	123.5	2342	113.5	2335	115.0
69000	2449	131.5	2449	120.0	2441	121.0
72000	2555	139.5	2555	127.5	2547	128.5
88270	3132	Failure				
104150					3684	Failure
109640			3891	Failure		
0.45 Max.	1409	69.0	1751		1658	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 NO. 5

Specimen: 4" x 8" cylinders

Curing: 7 days moist, 83 days air

Mix and Specimen No.		Date Tested	Dimensions, in.		90 Day Weight, lb.	Ultimate Load, lb.	Type of Break	Per cent Broken Aggregate	f'c, lb. per sq. in.
			Diam.	Height					
A	1	6-23-50	3.95	8.00	3.72	14510	Diag.	25	1180
	2		3.97	8.08	3.80	18080	Diag.	35	1460
	3		3.97	8.06	3.86	18040	Diag.	35	1460
B	1	6-21-50	4.00	8.15	4.27	27640	Diag.	80	2200
	2		3.99	8.06	4.22	22880	Diag.	80	1830
	3		3.95	8.06	4.21	27530	Diag.	90	2250
C	1	6-16-50	3.99	8.10	4.23	29790	Cone	80	2380
	2		3.98	8.00	4.18	31760	Diag.		2550
	3		3.96	8.10	4.23	31280	Diag.		2540
D	1	6-15-50	4.01	8.06	4.55	32940	Cone	75	2610
	2		3.97	8.12	4.51	35010	Cone		2830
	3		4.00	8.12	4.89	45400	Diag.		3610
E	1	6-26-50	3.98	8.10	4.44	34770	Diag.	80	2800
	2		3.99	8.08	4.47	32630	Diag.	80	2616
	3		3.99	8.10	4.41	36990	Diag.	80	2960
G	1	6-20-50	3.97	8.05	7.92	35120	Cone		2840
	2		4.00	8.04	8.03	37000	Diag.	1	2940
	3		4.01	8.06	7.74	35990	Diag.	1	2850

DATA ON NINETY DAY COMPRESSIVE STRENGTH TEST
TEST NO. 5 (Cont'd)

Specimen: 4" x 8" cylinders

Curing: 7 days moist, 83 days air

Mix and Specimen No.	Date Tested	Dimensions, in.		90 Day Weight, lb.	Ultimate Load, lb.	Type of Break	Per cent Broken Aggregate	f'c, lb. per sq. in.
		Diam.	Height					
H 1	6-19-50	4.00	8.02	5.55	38040	Diag.	35	3030
2		4.00	8.08	5.60	40570	Diag.		3230
3		4.01	8.06	5.67	37530	Diag.		2970

DATA ON FLEXURE TEST
TEST NO. 6

Specimens: 6"x6"x36" Curing: 28 days moist

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

DATA ON SONIC MODULUS TEST
TEST NO. 6_s

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

Mix	Date Tested	Depth, in.	Width, in.	Weight, lb.	"Range"* of Test	Dial Reading	Frequency cycles/sec.	E_s, 10^6 lb./in.^{2}
A	4-22-50	6.00	6.00	59.0	2	45.2	455	1.29
B	4-20-50	6.00	6.00	60.0	2	42.0	500	1.59
C	4-15-50	6.05	6.00	58.5	2	41.0	512	1.58
D	4-14-50	6.00	5.90	65.0	2	38.5	549	2.11
C_f	6-17-50	6.00	6.00	61.6	2	39.0	540	1.90
D_f	6-10-50	5.90	5.90	62.3	2	39.0	540	2.06
E	4-25-50	6.00	6.00	61.0	2	41.6	503	1.64
G	4-19-50	6.00	5.90	106.0	2	33.3	640	4.65
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 NO. 7

Specimens: 8" x 8" cylinders with 5/8" round deformed bars Curing: 7 day moist, 21 day air

Mix and Specimen No.		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,
A	1	4-22-50	8.00	4120	9390	Split	15.71	262	598
	2		8.08	4000	8310	Split	15.87	252	524
	3		8.00	4000	7430	Pull Out	15.71	255	473
B	1	4-20-50	8.08	5490	10360	Split	15.87	346	653
	2		8.08	5400	8690	Split	15.87	340	548
	3		8.02	5800	9690	Split	15.75	368	615
C	1	4-15-50	8.04	9120	11440	Split	15.79	578	725
	2		8.08	8130	10870	Split	15.87	512	685
	3		8.10	7850	12340	Split	15.91	493	776
D	1	4-14-50	8.06	7480	9440	Split	15.83	473	596
	2		8.08	8650	14010	Split	15.87	545	883
	3		8.05	8720	13480	Split	15.81	552	853
C_f	1	6-17-50	8.02	5240	9670	Split	15.75	333	614
	2		8.00	7210	10920	Split	15.71	459	695
	3		8.00	7630	12430	Split	15.71	486	791
D_f	1	6-10-50	7.95	9150	14460	Split	15.61	586	926
	2		7.98	8350	15670	Split	15.67	533	1000
	3		8.15	9560	9560	Split	16.01	597	597

DATA ON BOND TEST
TEST NO. 7 (Cont'd)

Specimens: 8" x 8" cylinders with 5/8" round deformed bars   Curing: 7 day moist, 21 day air

Mix and Specimen No.		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
E	1	4-25-50	8.06	7320	9870	Split	15.83	462	623
	2		8.04	6410	10350	Split	15.79	406	655
	3		8.10	4880	9410	Split	15.91	307	591
G	1	4-19-50	8.16	4930	17150	Split	16.03	308	1070
	2		8.05	4120	19020	Steel	15.81	261	1203
	3		8.01	6010	19110	Steel	15.73	382	1215
H	1	4-18-50	8.02	9370	19070	Steel	15.75	618	1211
	2		8.04	7420	18950	Pull Out	15.79	470	1200
	3		8.06	8850	17740	Split	15.83	559	1121

DATA ON ABSORPTION TEST
TEST NO. 9

Specimens: 4" x 8" cylinders

Curing: 7 days moist, 21 days air

Mix and Specimen No.	Dimensions: Diam., in.	Dimensions: Height, in.	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 Vol.
A 1	4.00	8.06	3.63	62.1	4.31	18.7	18.7
2	Poor	Surface	3.51		4.20	19.6	
3	3.99	8.08	3.59	61.5	4.27	18.9	18.7
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
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
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
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

DATA ON ABSORPTION TEST
TEST NO. 9 (Cont'd.)

Specimens: 4" x 8" cylinders

Curing: 7 days moist, 21 days air

Mix and Specimen No.		Dimensions		Oven Dry Wt., lb.	Oven Dry Unit Wt., lb./cu.ft.	24 Hour Immersion Wt., lb.	Absorption, per cent	
		Diam., in.	Height, in.				By Dry Wt.	By Vol.
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
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

DATA ON SHRINKAGE TEST
TEST NO. 10

Specimens: 3" x 3" x 11" bars

Curing: 7 days moist,
21 days air

Mix and Specimen No.		Length, in.			Shrinkage, per cent	
		1 Day	28 Days	Oven Dry	28 Day Average	Oven Dry Average
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		
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		
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		
D_f	1	9.9931	9.9912		0.027	
	2	10.0154	10.0126			
	3	10.0079	10.0043			
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		

DATA ON SHRINKAGE TEST
TEST NO. 10 (Cont'd.)

Specimens: 3" x 3" x 11" bars

Curing: 7 days moist, 21 days air

Mix and Specimen No.		Length, in.			Shrinkage, per cent	
		1 Day	28 Days	Oven Dry	28 Day Average	Oven Dry Average
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		
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.