Highway Materials Research Laboratory
132 Graham Avenue, Lexington 29, Kentucky

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TO: D. V. Terrell
Director of Research

You will recall the project in Montgomery County where a lean concrete base for a bituminous surface was placed last summer and, also, the laboratory work dealing with lean concrete mixes, which we initiated at your suggestion. These were carried out simultaneously late last summer and in the fall, and the results to date are contained in the attached report.

It is too early to judge the field project except to the extent that in most cases the strengths of specimens made in the field were somewhat lower than anticipated. However, as we have indicated before relatively low strengths are theoretically desirable because of the favorable shrinkage characteristics that go along with them. Viewed from the standpoint of thickness, the pavement now has a minimum depth of 11 inches where the lean concrete base was used, as opposed to 9 inches where the old concrete was surfaced.

There is some interest in the fact that the Wisconsin Highway Department tried some lean concrete base this past year, although in their case 8 inches of the lean mix was placed over an old concrete pavement that had been fragmented, and this was covered with $3\frac{1}{2}$ inches of bituminous concrete. That lean base contained no reinforcement, no joints except a longitudinal dummy cut at the close of the finishing operation. Mr. Collier mentions this work on page 3 of his report, and gives a reference so that those who are interested may find a more detailed account of the features in this Wisconsin pavement.

I am sure that we will look forward with interest to the reports of performance on our Montgomery County project as time goes on.

Respectfully submitted,

L. E. Gregg

L. E. Gregg
Assistant Director of Research

LEG:DDC
cc: Research Committee Members
Mr. Galbreath (3)

Commonwealth of Kentucky
Department of Highways

REPORT NO. 1

ON

AN INVESTIGATION OF LEAN CONCRETE MIXES AS BASE COURSES FOR BITUMINOUS SURFACES

by

S. T. Collier
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Lexington, Kentucky

January, 1951

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INTRODUCTION

Cement concrete bases for pavements with bituminous surfaces are, of course, not new. However, it has been many years since concrete has served this purpose except as a pavement itself ultimately resurfaced after years of service. The outstanding qualities of many roads which developed in this way were obvious without any particular tests, and for many years there apparently was no attempt to evaluate them. Probably structural value was first investigated in tests on airfields in Florida, Ohio and California. Although the original concrete pavements in all cases were heavy in comparison with highway sections, the results of the tests have some bearing on highway considerations. In "beefing up" rigid pavements that had failed under heavy loads and running subsequent tests under moving wheels, the Army Engineers* came to the conclusion that "--treatments of as little as 3 inches of asphaltic concrete give astounding structural benefits. Accelerated traffic tests have shown overlays of less rigid material to be so beneficial that original designs utilizing a rigid slab of moderate thickness covered with a flexible type surface can and probably will be in cost competition where conditions are favorable".

This information on structural qualities, the generally good appearance and service features of resurfaced concrete highway pavements, and the generally poor riding qualities of flexible pavements with water-bound macadam bases built during

*"Design of Rigid Pavements For Heavy Wheel Loads", by R.R. Phillippe, Head, Ohio River Division Laboratories, U.S. Corps of Engineers, Civil Engineering, V. 18, n. 2, p. 32, February, 1948.

the past few years created interest on the part of the Research Laboratory in the advantage that might be gained through concrete bases with bituminous surfaces. Several features that are relatively expensive in the construction of concrete pavements (such as elaborate finishing of the surface) could easily be modified or eliminated, and the most prominent of these which gained early attention was the cement content. Accordingly, a research project for studying lean concrete mixes was undertaken.

The use of concrete mixes with low cement content in the construction of portland cement concrete bases for bituminous surfaces has been recommended by the Asphalt Institute. Presumably these recommendations were substantiated by data collected from observations of a number of projects with this type of construction, whose records evidenced successful performances.

A desirable property of lean concrete, concomitant with the factor of economy, is its low coefficient of expansion -- a feature preferred over the richer mixes, provided adequate strength is obtained. Cracks may sometimes occur at a greater frequency, but the movement is of much less magnitude and hence, there is greater possibility such movement will not be reflected in the covering surface. This property should theoretically reduce tendencies toward faulting and buckling.

The design proportions ususally employed were expressed as 1:3:6 mixes (loose volumes) or leaner. This proportioning, converted to solid volumes, is approximately equal to four sacks of cement per cubic yard of concrete with the ratio of fine aggregate to total aggregate being 36 to 37 per cent by weight.

In addition to the usual method of concrete placement, it has been reported that concrete bases have been successfully constructed by spreading stiff concrete mixes loosely in place and compacting them by rolling under the usual type of two or three-wheeled roller.

During the past year the Wisconsin Highway Department* employed both methods in pavement reconstruction. In the first case the badly damaged existing pavement was fragmented and rolled, then covered with a 4-sack concrete base and 350 pounds per square yard of asphaltic concrete surface (in two courses). In the second case where the existing concrete pavement was in better than average condition, it was covered with an 8-inch slab of pug mill (2.4 sack) concrete. The concrete was placed with an asphalt paver and compacted with steel and pneumatic rollers and "immediately covered with two courses of asphaltic concrete".

The Kentucky Department of Highways, during the summer of 1950, also used a lean concrete mix for base construction and widening prior to surfacing with asphaltic concrete. This report is a presentation of data collected from this project and from a laboratory investigation of lean concrete conducted by the Research Laboratory.

*"Repaving For Very Severe Conditions", a report on a lean concrete slab and heavy bituminous blanket placed over an old concrete pavement in Wisconsin, where pavement service conditions are called severest in the state, Roads and Streets, v. 93, n. 11, p. 33, November, 1950.

FIELD PROJECT

The field portion of this study consisted of the making and testing of samples, recording of cracks in the lean concrete base, the observation of construction methods, and the evaluation of performance under traffic during the brief period which has elapsed since construction. This project was a 4.415 mile section of US 60 in Montgomery County on the Winchester-Mt. Sterling Road, and was designated as SP 87-117. Contractor for the job, which covered both the concrete base and bituminous surfacing, was A. W. Walker and Son.

The existing eighteen foot concrete pavement, built in 1924, was for the most part utilized as a base for the new bituminous surface. Some revisions in alignment and grade, were included as a means of eliminating the worst driving conditions along the road. Normally, the widening comprised two feet of additional width of new base on each side of the old slab, thus increasing the pavement to twenty-two feet. The revisions in alignment on four curves were accomplished by laying extra width of base on the inside of the curve only. These extra widths at points of maximum widening varied from 4.5 feet to 15.5 feet. Four major revisions - three alignment and one grade - involved new grade construction, thus, necessitating the replacement of sections of the old pavement with new full-width base and, of course, bituminous surface.

Weakened plane contraction joints were placed coincident with the joints in the existing slab (at intervals of 30 feet)

Fig. 1. Mt. Sterling-Winchester Road, U.S. 60, in Montgomery County. The two-foot widening in the foreground was placed on the North side of the old pavement, and contained no contraction joints. On the South side, contraction joints were placed coincident with joints in the existing slab at intervals of 30 feet. The old pavement was built in 1924, and had a 6-8-6-inch section.

in the widened edges on the south side, and at the same interval in the full width slab between Stations 190 +00 and 196 + 00. Joints, except for construction joints, were eliminated entirely on the widened edges on the north side and in the three remaining full width sections. Expansion joints were not included in this project.

The thickness of the widened slab was six inches and that of the full width sections was 6-8-6 inches, conforming with the cross-section of the existing pavement. Stone insulation about 2-inches in depth was placed beneath the new concrete on both the widening strips and the revisions.

The concrete was delivered by transit mixers from the batching plant at Mt. Sterling. The surface was struck off by a hand drawn vibrating screed and finished by hand float.

The concrete mix used in the base construction was designed with the following requirements:

Cement Factor - 3.5 sacks per cubic yard of concrete
Maximum Free Water - 9.75 gallons per sack of cement
Ratio of Fine Aggregate - 34 to 38 percent of total aggregate by weight
Entrained Air - 3 to 6 percent

The material used were air-entraining Portland cement, Ohio River sand and Size No. 36 crushed limestone.

The maximum free water of 9.75 gallons per sack was estimated. Since approximately the same quantity of water per cubic yard of concrete is needed to maintain a given consistency, irrespective of the cement content, the total mixing water of 9.75 gallons of water per sack for 3.5 sack concrete is equivalent to 5.75 gallons per sack for 6-sack concrete, or 34 gallons

Fig. 2. Placing the lean concrete mix in the widening strip. Note the wooden forms, and the 2-inch stone insulation course beneath the pavement grade. Concrete for both the widening strips and the revisions was delivered by transit mixers from Mt. Sterling. A hand-drawn vibrating screed was used to strike off the surface in the widening strips.

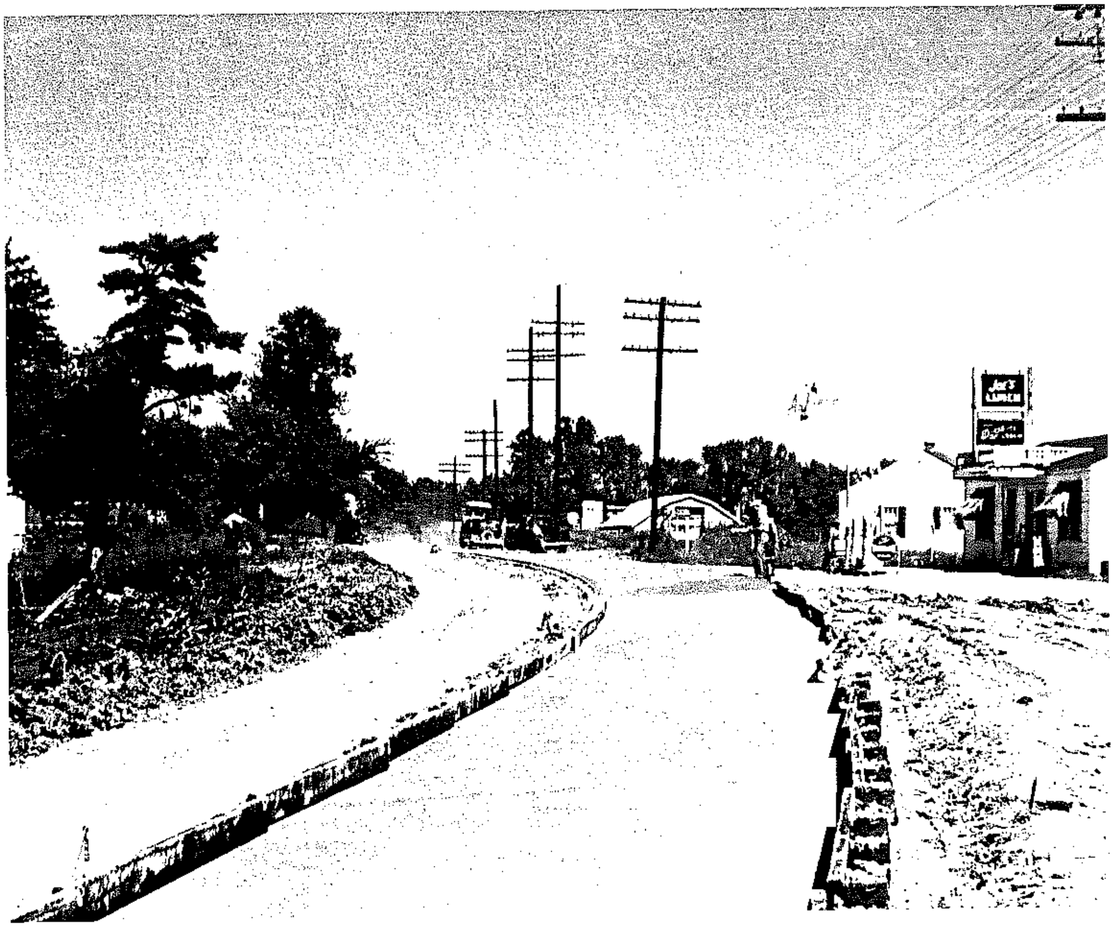


Fig. 3. Full-width revisions were placed in single lanes. This view shows one finished lane in the revision between Sta. 148+50 and Sta. 155+07. In the middle distance membrane curing compound is being placed on concrete that was finished a short time before. A vibrating screed operating lane-width across forms was used for striking off the concrete, after which the surface was finished by hand. At this particular location, and generally throughout the job on both revisions and widening strips, the hand finishing was carried much farther than necessary on a surface to be covered with a bituminous mix.

per cubic yard of concrete. Field reports indicated that average mixing water was approximately 8.5 gallons per sack - an underrun of some 30 pounds.

Construction of the concrete base was carried on between the dates of July 18 and September 10, 1950. Bituminous surfacing was started in October and completed before the end of the construction season.

LABORATORY PROJECT

The investigation conducted in the Research Laboratory was expanded to a study of fifteen mix designs for air-entrained concrete with variables in cement content, and type and size of coarse aggregates. The purpose was to arrive at a comparative evaluation of these several mix designs with respect to their cement contents and aggregate combination as affecting strength, workability and other characteristics. Mixes were designed for three cement factors of 3.5, 4.0 and 6.0 sacks of cement per cubic yard with each of the five coarse aggregates. The coarse aggregates were river gravel (sizes No. 6 and No. 36), and crushed limestone, (sizes No. 6, No. 36, and a combination of No. 2, No. 3 and No. 6). For convenience, the latter will be identified throughout the report as No. 236.

The design method proposed by the National Crushed Stone Association was followed to arrive at the mix proportions, employing the b/bo factor as defined by the densities of the compacted aggregates. This method is essentially the same as that employed by the Highway Department except that it offers a more specific indication of the fine aggregate requirement as related to the void space in the compacted coarse aggregate. The various percentage ratios were as follows;

Coarse Aggregate	Ratio of Fine Aggregate by Wt. of Total Aggregate
No. 6 River Gravel	35 percent
No. 36 River Gravel	30 percent
No. 6 Crushed Limestone	38 percent
No. 36 Crushed Limestone	34 percent
No.236 Crushed Limestone	29 percent

These percentages were satisfactory according to observations made in the laboratory, but are probably somewhat lower than would normally be preferred in the field.

Inasmuch as an increase in the spread of the nominal sizes of a given aggregate (with proper distribution) results in a greater density (or lesser void space and surface area) then the quantity of fine aggregate needed should be decreased. With a reduction of fine aggregate the total surface area per unit volume is further reduced. With these conditions prevailing it seems reasonable to assume that concrete of higher strength should be the result - particularly for mixes with low cement content. Thus, concrete with the extremely coarse aggregate (No. 236) was included in this study for the purpose of investigating the feasibility of its use in slab construction.

A single brand of plain portland cement, Type I, was used for the entire laboratory project. Air was entrained by the addition ofneutralized vinsol resin to the mixing water.

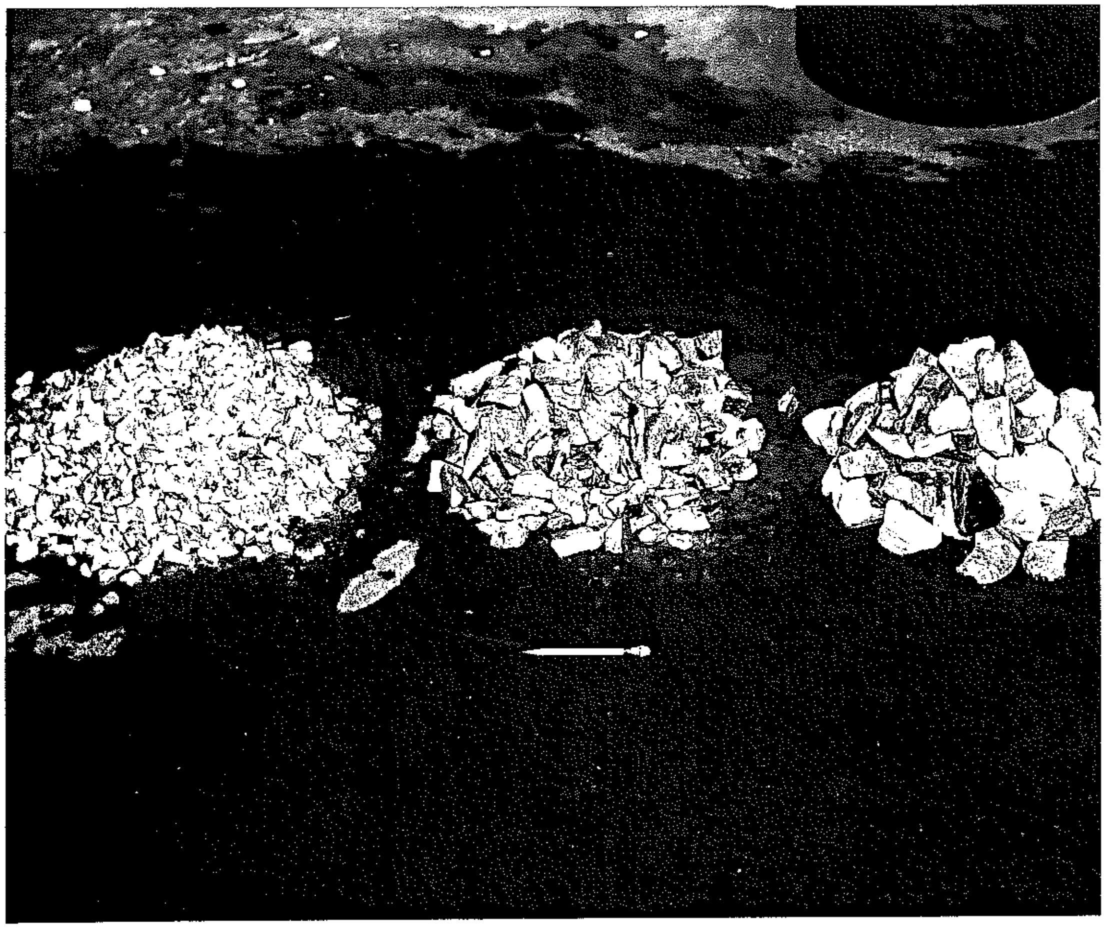


Fig. 4. Three sizes of crushed limestone coarse aggregates used in the laboratory tests. From left to right are No. 6, No. 3, and the 2-to $2\frac{1}{2}$-inch stone which is an extraction from No. 2. In combination these made the No. 236 aggregate which was investigated for possibilities of increasing strength at the lower cement factor and still retain the advantages of reduced shrinkage characteristics.

The physical properties of fine and coarse aggregates are given in Table I. The fine aggregate was a pit sand from Cleves, Ohio; the coarse aggregates were two types: Ohio River gravel from Louisville, and crushed limestone from Lexington.

The coarse aggregates were reprocessed at the laboratory to standard sizes closely approximating the median gradation for sizes No. 6 and No. 3 for both the gravel and the limestone, as well as a stock of one-sized crushed stone between the 2½-inch and 2-inch sieve sizes. All sizes were stored separately and introduced to the mixes separately, but in proportions to conform to the computed aggregate gradations given in Table I. All aggregates were stored under moist conditions.

Each mix required three batches of 1.9 cubic-feet to yield the quantity of concrete needed for molded test specimens and plastic concrete tests. Tests specimens made from each batch consisted of the following: one each 6 x 12-inch and 8 x 16-inch cylinders; and 5 x 6 x 20-inch and 6 x 6 x 22-inch beams (except for the series containing the No. 236 size aggregate). For the latter series the 6 x 12-inch cylinders were eliminated and the 5 x 6-inch beams were replaced by 6 x 6-inch beams.

Tests for slump, air content, and unit weights were made for each batch.

Although the 8 x 16-inch cylinders were required only for the concrete containing aggregate exceeding two-inches in size, they were cast for all series for the purpose of strength comparisons between the large and small cylinders. As will be

TABLE I

PHYSICAL PROPERTIES OF AGGREGATES

Coarse Aggregates

Type	River Gravel		Crushed Limestone		
Standard Size Identification	No. 6 A	No. 36 B	No. 6 C	No. 36 D	No. 236 E
Gradation					
Sieve Sizes	Percent Passing				
2½ inches	-	-	-	-	100
2 inches	-	100	-	100	85
1½ inch	-	83	100	77	72
1 inch	100	55	99	50	55
3/4 inch	70	35	75	37	41
1/2 inch	40	20	35	18	19
3/8 inch	20	10	16	8	9
No. 4	0	0	0	0	0
Compacted Unit Wt. Pounds Per Cu. Ft.	105.5	109.4	96.5	103.6	107.0
Bulk Sp. Gr. S.S.D.	2.66	2.66	2.72	2.73	2.73
Bulk Sp. Gr. O.D.	2.62	2.62	2.69	2.70	2.70
Pct. Absorption	1.9	1.9	1.1	0.8	0.8

Fine Aggregate (Concrete Sand)

Gradation		
Sieve Size	Percent Passing	
No. 4	100	Bulk Sp. Gr. S.S.D. - 2.67
8	87	Bulk Sp. Gr. O.D. - 2.62
16	60	Percent Absorption - 1.9
30	33	Fineness Modulus - 3.11
50	8	
100	1.5	
200	0.5	

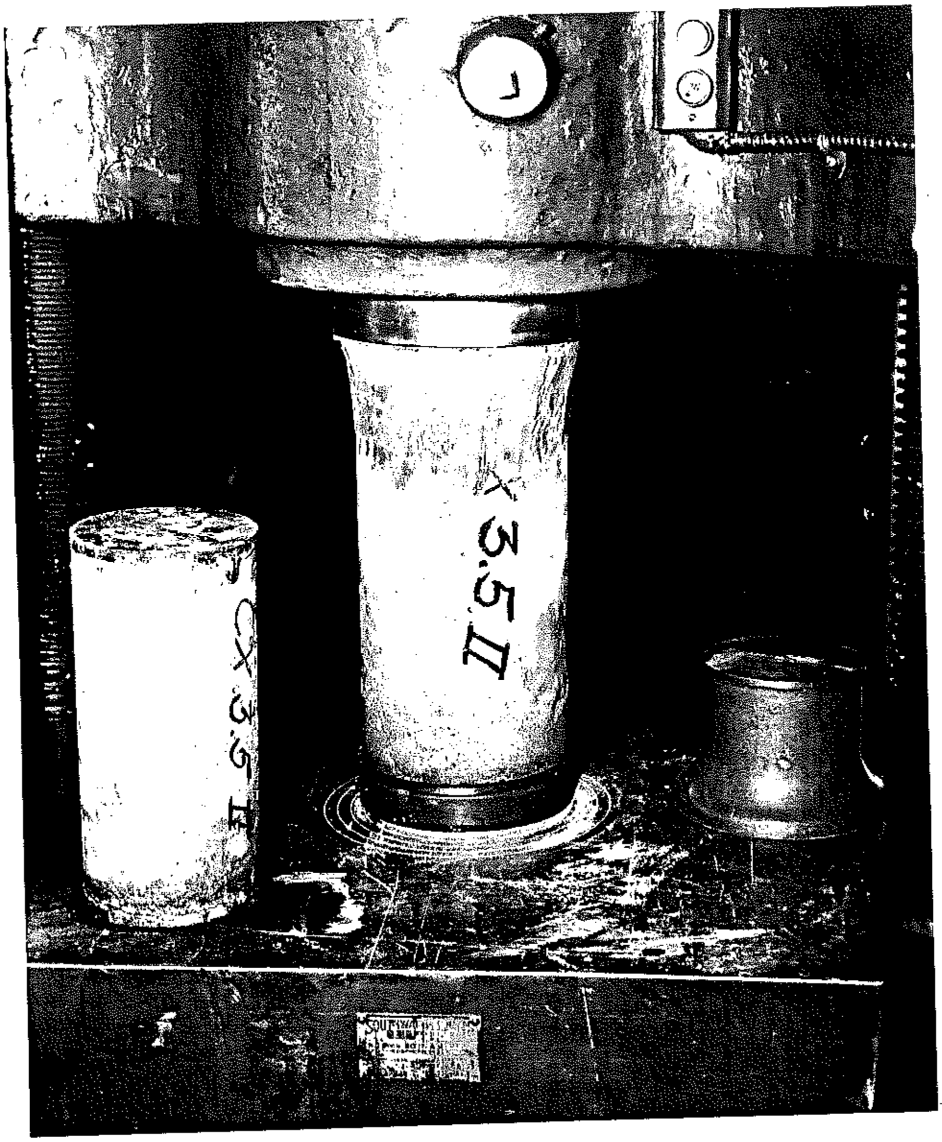


Fig. 5. Two sizes of specimens were made for laboratory tests in order to accommodate the aggregates greater than 2 inches in size. On the left is 6 x 12-inch cylinder commonly used with the aggregates smaller than 1½ inch in size, and in the testing machine ready for test is an 8 x 16-inch cylinder. Comparable beam samples were 5 x 6 x 20-inches and 6 x 6 x 22 inches in size. The small beams and cylinders were eliminated when the aggregate exceeded 1½-inches top size.

noted under the discussion of results there was an appreciable difference in the respective values.

All specimens were moist cured until the date of test. Three beams from each series were selected for seven-day tests while the remaining specimens were tested at twenty-eight days.

RESULTS AND OBSERVATIONS

Field Specimens.

The tests for the field specimens resulted in somewhat lower strength than was anticipated. This is particularly true for specimens tested in compression. The test results are compiled in Tables II, IV, V, and VI, and in Figures 7, 8, and 9; (all except Table II being placed in the back of the report).

The values selected as expected strengths of 28-day concrete, 2200 pounds per square inch in compression and 475 pounds per square inch in flexure, were based on advance laboratory test results with allowances made for field conditions. By comparison of laboratory specimens from the same concrete, the average strengths of the 7-day beams were in general from 80 to 85 percent of those of the 28-day beams. Thus concrete, having beam strengths of 475 pounds per square inch at 28 days, should have strengths approaching 400 pounds per square inch at 7 days. The strengths of all field beams reported, are plotted in order of their magnitude, in Fig. 7. It is noted that 64.3 percent of the beams tested at 7 days are above 400 pounds per square inch, and 64.5 percent of those tested at 28 days are above 475 pounds per square inch. The average strength are 438 and 506 pounds per square inch respectively for the 7-day and 28-day specimens - a ratio of 86 to 100.

The test results of the field cylinders reported are given in Table V and Fig. 8. Only 22 percent obtained the specified strength of 2200 pound per square inch. Broken down in groups with respect to strength in pounds per square inch, 28 percent were between 1200 and 1500; 34.5 percent were between 1500 and 2000; and 15.5 percent were between 2000 and 2200. These results are more inconsistent and more widespread than normally would be expected, and were less favorable than were those for the beams.

Samples were made by representatives of the Research Laboratory for strength comparisons as influenced by curing conditions. (See Table II) A total of nine beams and nine

TABLE II

Strengths of Field Specimens
Subjected to Various Curing Conditions

Curing Medium	Moist Room	Damp* Soil	Air	Average By Batch
Batch No.	Moduli of Rupture of Beams - P.S.I.			
1	495	435	435	455
2	570	630	555	585
3	420	450	315	395
Average	495	505	435	
	Compressive Strength of Cyls. - P.S.I.			
1	2115	1970	1890	1990
2	2325	2130	2050	2170
3	1450	1475	1290	1405
Average	1965	1860	1745	

*Membrane treated face exposed to air.

cylinders were made three for each type of curing from each of three separate batches. All were cured 28 days, the first group in the moist room at 70°F, the second in damp soil with the membrane treated surface exposed, and the third group exposed to natural atmospheric conditions for the entire 28 days. No special significance can be attached to the strength differentials as influenced by curing conditions, although the strengths of specimens cured in the moist room and in damp soil were, as a rule, higher than those cured in air. There is a wider disparity among the batches themselves than among the curing conditions.

The compressive strengths of the cores, (shown in Table VI and Fig. 9) were slightly more uniform and slightly higher on the average than those of the cylinders. Although 37.5 percent tested below 2200 pounds per square inch, the average value was 2213 pounds per square inch.

Laboratory Specimens.

Mix data and results of strength tests for laboratory specimens are compiled in Tables VII A through VII E. The slumps and amounts of mixing water were reasonably uniform for all mixes, but there were some appreciable variations in the percentages of entrained air. In the series where the variations were relatively wide the increases in air content were reflected in the compressive strengths of cylinders, but little or no influence was indicated in the flexural strengths.

The strengths of the individual specimens are represented in Fig. 10 and 11 in groups as defined by the cement content, aggregate combination, and specimen type. For better comparison of the concrete with aggregate variables, the average strengths for the several groups are represented in Figs. 12 and 13 in groupings related to their cement content and specimen types.

All laboratory beams were broken under third-point loading as opposed to center loading for the field beams.

With all conditions, except the loading methods, being equal, the moduli of rupture of two beams have a definite numerical relationship. In the case of beams of six inches in depth and supported over 18 inches of span, the moduli of rupture for the one tested under center loading has a numerical value of 1.25 greater than that for another tested under third-point loading; although the load required to break the latter would be approximately 1.2 greater than that for the former. These relationships are theoretical, but they have been substantiated empirically through an investigation of concrete specimens in the laboratory several years ago.

In Fig. 12 the solid line bars represent the average of the moduli of rupture for beams tested under third-point loading. The dashed-line extensions represent the estimated results, had they been tested under center loading. This conversion permits a better comparison between the laboratory and the field beams. The average strengths of the laboratory beams (those made with the 3.5 sack cement factor and No. 36 crushed limestone) exceeded that of the field by 18 and 25 percent respectivelly for the 7-day and 28-day tests. Undoubtedly this was due, largely, to curing conditions.

Since there were no 6x12-inch cylinders made with concrete containg the No. 236 aggregates, the related values were estimated on the basis of the average of strength differentials between the two sizes of companion specimens, (see Table III), and these were represented in Fig. 13 by the

TABLE III

Percentages of Average Compressive Strengths of 8 x 16-inch Cylinders as Related to Companion 6 x 12-inch Cylinders.

Cement Factor	6.0 Sacks Per cu. yd.	4.0 Sacks Per cu. yd.	3.5 Sacks Per cu.yd.	Avg. by Aggregate Type
Aggregate	Strength Percentages			
No. 6 Gravel	87.2	81.9	94.6	87.9
No. 36 Gravel	86,1	88.9	78.6	84.5
No. 6 Cr. L. S.	78.9	94.6	91.1	88.2
No. 36 Cr. L.S.	88.8	89.1	86.7	88.2
Ave. by C. F.	85.3	88.6	87.6	87.2

dashed line bars. The average of the compressive strengths of the cylinders made of 3.5-sack concrete and containing the No. 236 crushed limestone was the lowest of all the groups made in the laboratory; but, exceeded by 16 percent the average for the field cylinders.

In general, there was no particular effect indicated on the strengths of the concrete as influenced by the type of aggregate. No one aggregate was especially superior or inferior in overall performance. The concrete mixes containing the No. 6 crushed limestone coarse aggregate may have held a slight strength advantage for all conditions as indicated by the results from this particular study.

The strength relationships among the several series of mixes are plotted with respect to their cement contents in Figures 14 and 15. These graphs add emphasis to the inconsistencies resulting among the strengths acquired by the different mixes. The variations are somewhat less widespread among the 3.5-sack mixes than in those of the 4-sack and 6-sack mixes. Also, in the majority of cases, there is a tendency for the 4-sack concrete to obtain strengths exceeding a "straight line" relationship for the three concretes.

Another development contrary to expectations, was the relationship between the compressive strengths of the 6 x 12-inch and the 8 x 16- inch cylinders. In every case the larger cylinders broke at a lower unit stress than their companion cylinders with smaller dimensions.

This relationship is shown in Table III for each series excepting those containing the No. 236 crushed limestone. The values are expressed as the percentage of the average strength attained by the 8 x 16 cylinders as compared to the average strengths of the 6 x 12 inch cylinders of the same concrete. These percentages are spread over a range of from a minimum of 78.6 percent to a maximum of 94.6 percent, but the average, with regard to either cement factors or aggregate types, do not vary greatly. The overall average gives a ratio of 87 for the 8 x 16 cylinders to 100 for the 6 x 12 cylinders.

Crack and Joint Survey

A crack and joint survey was made prior to the beginning surfacing operations and this condition is presented in Table VIII and Fig.16. Table VIII is a tabulation summarizing the frequency and location of cracks for the widened-edge portion only.

In the widened edges on the left side, in which contraction joints were not included, the number of cracks occurred at frequencies varying at rates of 1.0 to 10.5 cracks per one hundred feet. The average was 3.7 cracks per one hundred feet for the project. The greatest concentration of cracks formed was between stations 101 + 69 and 102 + 36, where the average interval was two to three feet.

On the right side, in which contraction joints were included at intervals of 30 feet, the total number of cracks was approximately 40 percent of that for the left side. However, in several instances, crack frequencies were as great as that for the left side - varying at the rates of from 0.1 to 9.9 cracks per 100 feet, and greater in some short intervals.

Where extra widening, exceeding four feet, was constructed for curve revisions, no cracks at all were found in the sections with contraction joints. In the sections that had no contraction joints, the average crack interval was only about one per 100 feet, and this condition was quite uniform.

Cracks and joints for the full width pavement sections are drawn in plan in Fig. 16. There were no transverse joints other than construction joints in the first three of these full width revisions; the fourth had weakened plane joints at 30-foot intervals.

Fig. 6. Finished concrete base on the full-width revision between Sta. 113+77 and Sta. 124+11, about three weeks after placement. At that time only two cracks had developed throughout the 1000-foot section.

Only two cracks were found in the first revision, (Sta. 113+77 to Sta. 124+11.4) and they were in the two end sections that were constructed at an early date to facilitate reopening the road to traffic. The construction of the base in the intermediate portion of this revision (Sta. 115+98 to Sta. 122+50) was delayed awaiting the completion of the grade, and was not open to traffic at the time of the crack survey.

The crack conditions in revisions 2 and 3 (Sta.134+09 to Sta. 141+83; and Sta. 148+50 to Sta. 155+07) were comparable, the former being only slightly better than the latter. The crack frequencies averaged 1.4 and 1.5 cracks per 100 feet respectively.

In the grade revision (Sta. 189+96 to Sta. 196+56), where weakened plane joints were placed, there were only two cracks - one across the full width of the slab,and one only half width.

So far as the full width revisions are concerned, the majority of the cracks either extended over the full width of the slab or adjoined construction joints. They were relatively straight, and with few exceptions were normal to the centerline. At the time of the survey, the crack openings were very slight, in some cases barely visible.

Supplementary inspection of the completed surface was planned, but weather conditions prevented this being done in detail up to this time.

TABLE IV

Moduli of Rupture of Field Beams

Beam No.	Location Sampled: Station to Station	Location Sampled: Lane	Age Tested Days	Modulus of Rupture: 7 Day P.S.I.	Modulus of Rupture: 28 Day P.S.I.
2	2+50 - 15+50	Rt.	28		500
5	34+50 - 56+10	Rt.	7	490	
6	34+50 - 56+10	Rt.	28		475
7	56+10 - 76+50	Rt.	7	488	
8	56+10 - 76+50	Rt.	28		750
9	76+50 - 102+50	Rt.	7	600	
10	76+50 - 102+50	Rt.	28		495
11	102+50 - 142+40		7	492	
12	126 - 142	Rt.	7	550	
13	142+40 - 177+25		7	480	
14	142+40 - 177+25	Rt.	28		517
15	0+00 - 23+45	Lt.	7	350	
16	117+25 - 201+45	Rt.	28		483
17	0+00 - 23+45	Lt.	7	350	
18	0+00 - 23+45	Lt.	28		617
19	0+00 - 40+50	Lt.	7	500	
20	23+45 - 40+50	Lt.	28		517
21	45+05 - 57+20	Lt.	7	320	
22	40+50 - 57+20	Lt.	28		508
23	42+85 - 45+05	Lt.	7	380	
24	42+85 - 89+61	Lt.	28		550
25	111+80 - 130+70	Lt.	7	467	
26	89+50 - 130+70	Lt.	28		550
28	130+70 134+00 142+00 147+20 155+06 180+50	Lt.	28		567

TABLE ~~VI~~ IV (CONTINUED)

Moduli of Rupture of Field Beams

Beam No.	Location Sampled: Station to Station	Location Sampled: Lane	Age Tested Days	Modulus of Rupture 7 Day P.S.I.	Modulus of Rupture 28 Day P.S.I.
29	130+70 - 147+20	Lt.	7	453	
30	196+60 - 217+43	Lt.	28		600
31	217+43 - 236+36	Lt.	7	366	
33	64+08 - 69+50	Lt.	7	350	
35	148+43 - 155+07	FWR Rt.	7	400	
37	139+25 - 141+79	FWR Rt.	7	365	
38	139+25 - 141+79	FWR Rt.	28		400
39	137+75 - 141+79	FWR Lt.	7	333	
40	137+35 - 141+79	FWR Lt.	28		383
41	134+00 - 139+35	FWR Rt.	7	471	
42	148+50 - 155+00	FWR	28		450
43	134+00 - 138+54	FWR Lt.	7	457	
44	134+00 - 138+54	FWR Lt.	28		500
45	190+50 - 194+00	FWR Lt.	7	430	
46	190+50 - 194+00	FWR Lt.	44		450
47	190+50 - 193+25	FWR Rt.	7	455	
48	190+50 - 195+25	FWR Rt.	28		591
49	114+05 - 116+00	FWR	7	550	
50	114+05 - 116+00	FWR	33		383
51	122+50 - 124+50	FWR	7	444	
53	121+00 - 122+50	FWR Rt.	7	408	
54	121+00 - 122+50	FWR Rt.	30		454
55	116+00 - 122+50	FWR	7	445	
56	116+00 - 122	FWR	28		384
	Averages			438	506

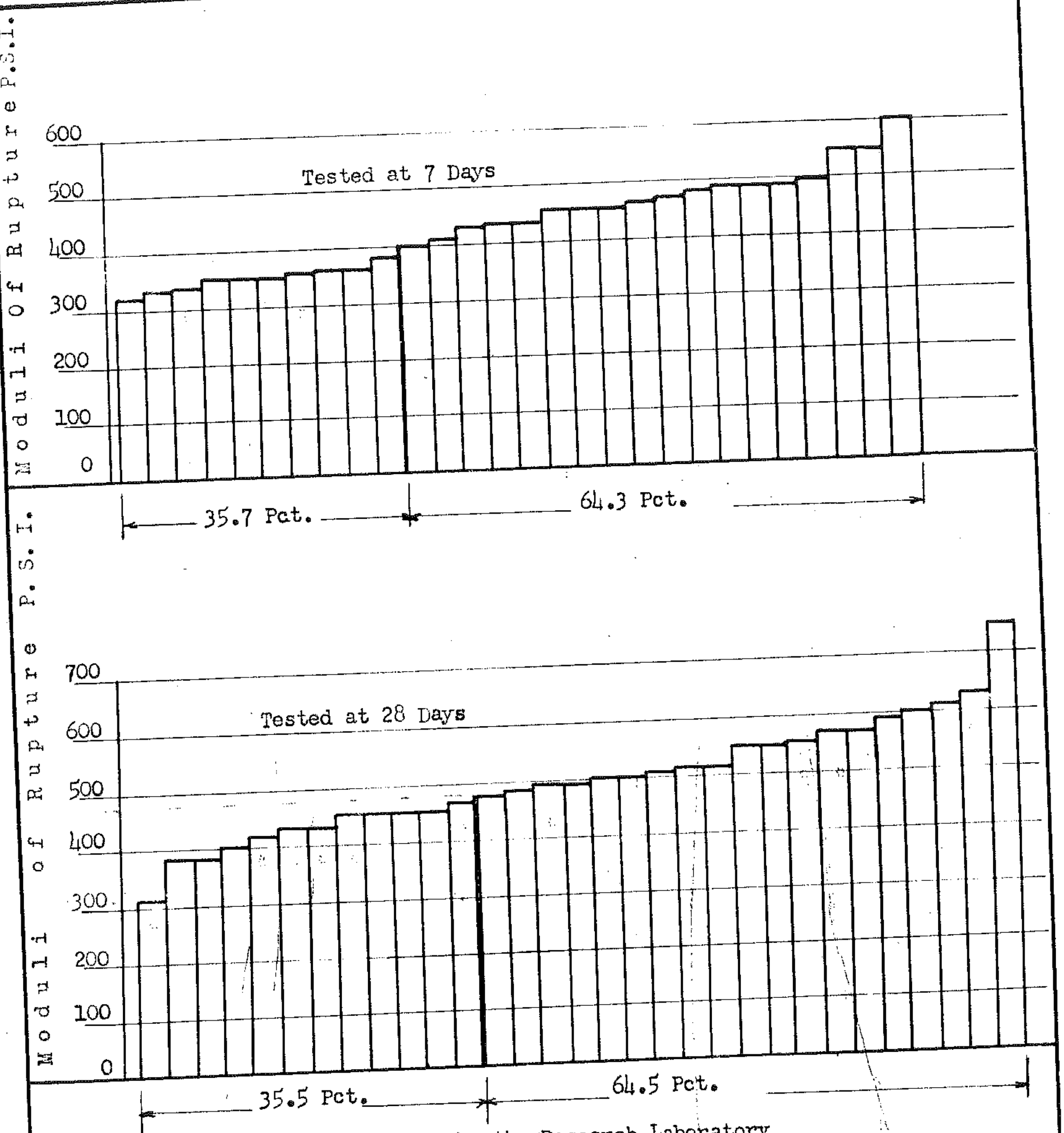
FWR = Full Width Revision

TABLE V

Compressive Strength of Field Cylinders

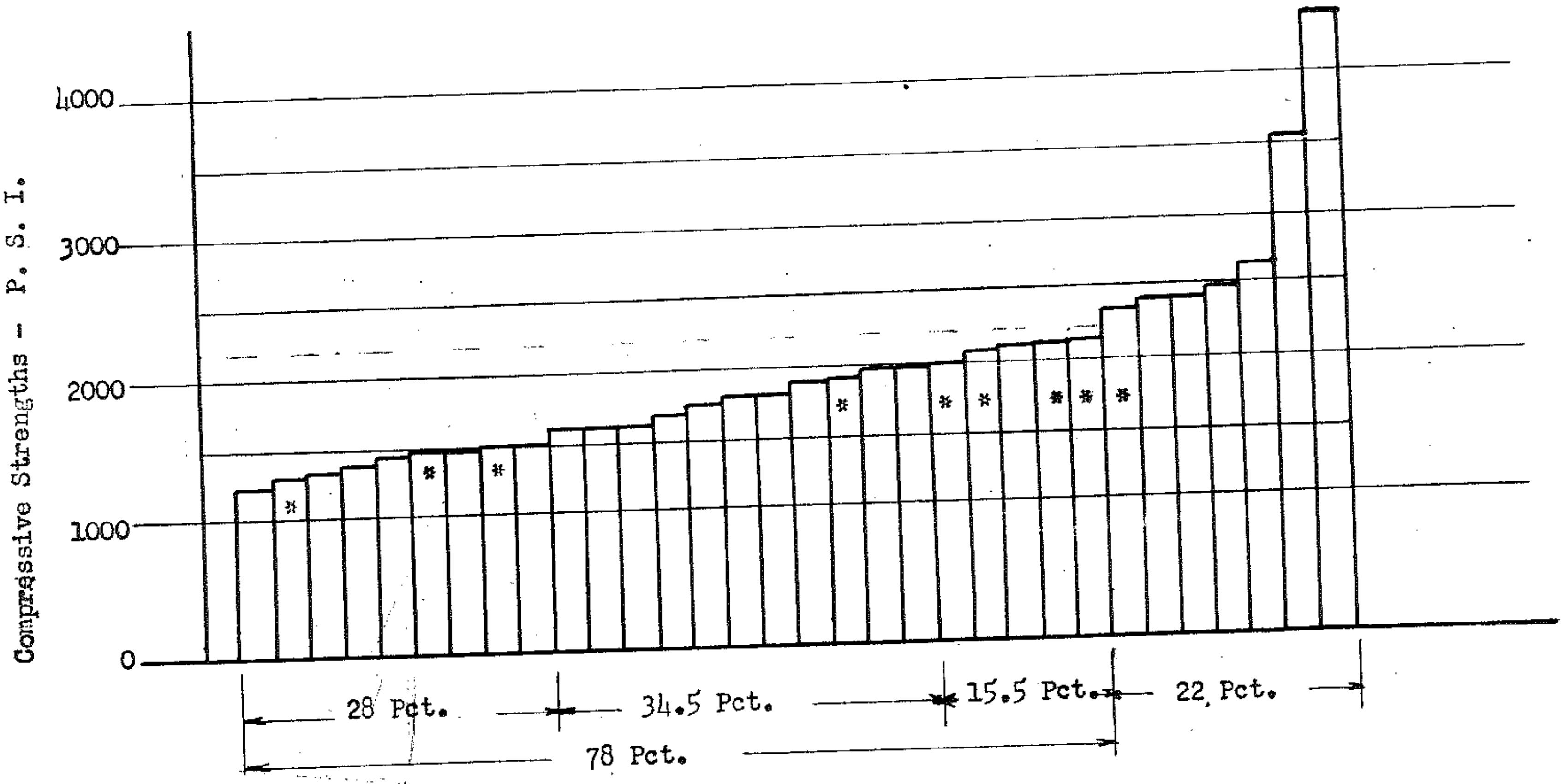
Cyl. No.	Location Sampled: Station to Station	Location Sampled: Lane	Age Tested Days	Compressive Strength P.S.I.
1	2+50 - 15+50	Rt.	35	1770
2	16+00 - 34+50	Rt.	33	1310
3	34+50 - 56+10	Rt.	32	1735
4	56+10 - 76+50	Rt.	35	2620
5	76+50 - 102+50	Rt.	34	1770
6	126 - 142	Rt.	33	1875
7	142+40 - 177+25	Rt.	32	1200
8	177+25 - 201+45	Rt.	31	1590
9	0+00 - 23+45	Lt.	28	2405
10	23+45 - 40+50	Lt.	34	4420
11	40+50 - 57+20	Lt.	33	1450
12	42+85 - 89+50	Lt.	32	1415
13	89+50 - 130+70	Lt.	42	1945
15	196+50 - 217+43	Lt.	40	2475
19	139+25 - 141+79	FWR Rt.	35	1485
20	134+00 - 137+35	FWR Rt.	33	1345
21	148+50 - 155+00	FWR	36	2097
22	134+00 - 138+50	FWR Lt.	35	2405
23	190+50 - 194+00	FWR Lt.	44	1590
24	190+50 - 193+25	FWR Rt.	42	1660
25	112+65 - 114+05	FWR	33	1945
27	122+50 - 121+00	FWR Rt.	42	1595
28	116+00 - 122+50	FWR	40	3537
			Average	1984

FWR = Full Width Revision



* Sampled and Tested by the Research Laboratory

Fig. 7 - Moduli of Rupture of 7-Day and 28-Day Concrete Field Beams.



* Sampled by Research Laboratory

Fig. 8 - Compressive Strengths of Concrete Field Cylinders

Table VI - Compressive Strenths of Cores

Core No.	Station	Location	Height Inches	Age Days	Strength P.S.I.
1	1 ≠ 00	Right	7	64	1763
2	2 ≠ 50	"	7-1/4	64	1903
3	3 ≠ 04	"	6-1/4	63	2186
4	4 ≠ 00	"	7-1/4	63	1438
5	5 ≠ 50	"	6-3/4	63	2256
6	16 ≠ 10	"	7	61	2520
7	20 ≠ 00	"	7-1/2	61	2609
8	20 ≠ 60	"	7	61	3032
				Average	2213

Percentage of cores that tested under 2200 P.S.I. = 37.5

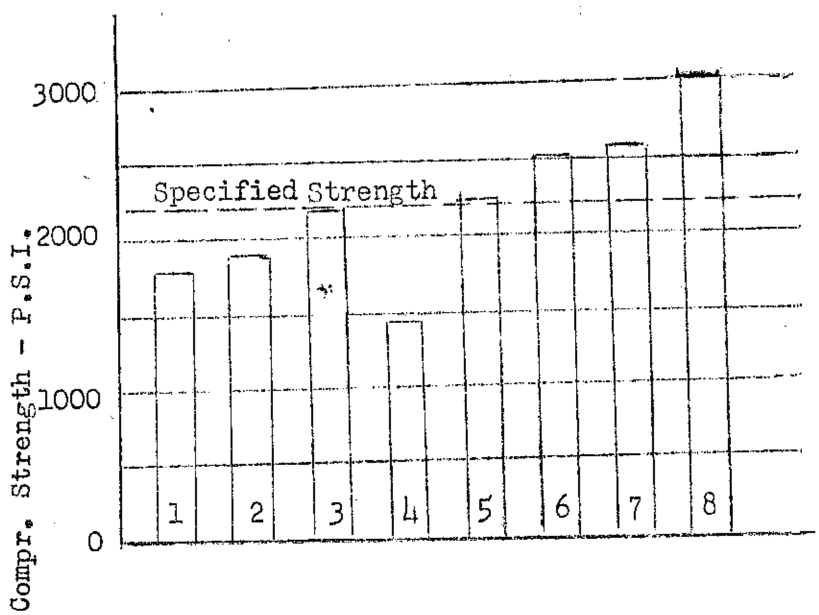


Fig. 9 - Compressive Strengths of Cores

TABLE VII-A. Concrete Mix Data and Strength Test Results

Coarse Aggregate - No. 6 Ohio River Gravel

Series	Batch No.	Design C.F. Sacks/C.Y.	Design W/C Gals./Sk.	Actual W/C Gals./Sk.	Slump inches	Entrained Air per cent	Compressive Strength 6x12 Cyls. P.S.I.	Compressive Strength 8x16 Cyls. P.S.I.	Moduli of Rupture *1 7-day Beams P.S.I.	Moduli of Rupture *1 28-day Beams P.S.I.
A-6	1	6.0	4.9	5.3	2-1/2	4.0	5005	4235	630	690
	2	6.0	4.9	5.0	2	4.9	4560	3990	630	635
	3	6.0	4.9	5.0	1-3/4	4.7	4870	4360	690	660
	Ave.						4810	4195	650*2	660
	Estimated for Center Loading								810	825
A-4	1	4.0	7.35	7.5	1-3/4	2.9	3380	3035	500	585
	2	4.0	7.35	7.5	2-5/8	3.1	3270	2605	460	560
	3	4.0	7.35	7.35	2-1/2	5.0	3090	2340	480	575
	Ave.						3245	2660	480	575
	Estimated for Center Loading								600	715
A-3.5	1	3.5	8.5	8.0	2-3/4	4.9	2290	2135	350	510
	2	3.5	8.5	8.5	2	3.2	2705	2540	410	485
	3	3.5	8.5	8.25	1-3/4	4.1	2660	2565	350	550
	Ave.						2550	2415	370	515
	Estimated for Center Loading								460	645

*1. Third Point Loading.

*2. Tested at 9 days.

TABLE VII-B. Concrete Mix Data and Strength Test Results

Coarse Aggregate - No. 36 Ohio River Gravel

Series	Batch No.	Design C.F. Sacks/C.Y.	Design W/C Gals./Sk.	Actual W/C Gals./Sk.	Slump inches	Entrained Air per cent	Compressive Strength 6x12 Cyls. P.S.I.	Compressive Strength 8x16 Cyls. P.S.I.	Moduli of Rupture 7-day Beams P.S.I.	Moduli of Rupture 28-day Beams P.S.I.
B-6	1	6.0	4.9	4.9	2-1/2	3.2	4700	4130	560	685
	2	6.0	4.9	4.9	2	2.9	4775	3860	580	585
	3	6.0	4.9	4.7	2	3.8	4490	4030	600	640
	Ave.						4655	4005	580	635
	Estimated for Center Loading								725	795
B-4	1	4.0	7.35	6.9	2-1/2	5.0	3130	2815	460	585
	2	4.0	7.35	6.8	1-3/4	3.6	3640	3395	480	615
	3	4.0	7.35	6.8	2	3.9	4065	3415	500	640
	Ave.						3610	3210	480	615
	Estimated for Center Loading								600	770
B-3.5	1	3.5	8.0	7.8	1-7/8	2.8	3805	2315	500	600
	2	Discarded - batching error			-	-	-	-	-	-
	3	3.5	8.0	7.8	2-3/8	3.4	2680	2390	420	515
	Ave.						2990	2350	460	560
	Estimated for Center Loading								575	700

TABLE VII-C. Concrete Mix Data and Strength Test Results.

Coarse Aggregate – No. 6 Crushed Limestone

Series	Batch No.	Design C.F. Sacks/C.Y.	Design W/C Gals./Sk.	Actual W/C Gals./Sk.	Slump inches	Entrained Air per cent	Compressive Strength 6x12 Cyls. P.S.I.	Compressive Strength 8x16 Cyls. P.S.I.	Moduli of Rupture 7-day Beams P.S.I.	Moduli of Rupture 28-day Beams P.S.I.
C-6	1	6.0	5.25	5.25	2-1/2	3.0	5030	4050	700	800
	2	6.0	5.25	5.10	2-1/4	4.1	5020	4110	660	800
	3	6.0	5.25	5.15	2-1/2	5.0	4530	3315	790	775
	Ave.						4860	3825	725	780
	Estimated for Center Loading								905	990
C-4	1	4.0	Discarded	–	–	–	–	–	–	–
	2	4.0	8.0	7.8	2-1/8	4.2	3535	3190	430	710
	3	4.0	8.0	7.8	2	4.0	3150	3135	490	675
	Ave.						3350	3165	460	690
	Estimated for Center Loading								575	865
C-3.5	1	3.5	9.0	9.0	2-1/2	2.8	2725	2375	450	515
	2	3.5	9.0	8.8	2	3.1	2475	2320	340	575
	3	3.5	9.0	8.8	2-3/4	3.3	2695	2505	380	590
	Ave.						2630	2400	390	560
	Estimated for Center Loading								490	700

TABLE VII-D. Concrete Mix Data and Strength Test Results.

Coarse Aggregate - No. 36 Crushed Limestone

Series	Batch No.	Design C.F. Sacks/C.Y.	Design W/C Gals./Sk.	Actual W/C Gals./Sk.	Slump inches	Entrained Air per cent	Compressive Strength 6x12 Cyls. P.S.I.	Compressive Strength 8x16 Cyls. P.S.I.	Moduli of Rupture 7-day Beams P.S.I.	Moduli of Rupture 28-day Beams P.S.I.
	1	6.0	5.17	4.9	2-1/2	4.5	4105	3515	650	625
D-6	2	6.0	5.17	4.9	2	4.3	4295	3750	600	665
	3	6.0	5.17	4.9	2	4.3	3880	3645	700	645
	Ave.						4095	3635	650	645
	Estimated for Center Loading.								810	805
	1	4.0	8.0	7.3	2	2.9	4030	3565	580	690
D-4	2	4.0	8.0	7.3	2-1/2	3.0	4015	3395	550	685
	3	4.0	8.0	7.2	2	3.2	3645	3465	580	775
	Ave.						3900	3475	570	715
	Estimated for Center Loading								710	895
	1	3.5	9.14	8.0	2-1/2	3.6	2465	2035	430	590
D-3.5	2	3.5	9.14	8.1	2-1/2	3.8	2440	2180	390	460
	3	3.5	9.14	8.2	2-3/4	4.6	2015	1790	410	475
	Ave.						2305	2000	410	510
	Estimated for Center Loading								510	635

TABLE VII -E. Concrete Mix Data and Strength Test Results

Coarse Aggregate - No. 236 Crushed Limestone.

Series	Batch No.	Design C.F. Sacks/C.Y.	Design W/C Gals./Sk.	Actual W/C Gals./Sk.	Slump inches	Entrained Air per cent	Compressive Strength 6x12 Cyls. P.S.I.	Compressive Strength 8x16 Cyls. P.S.I.	Moduli of Rupture 7-day Beams P.S.I.	Moduli of Rupture 28-day Beams P.S.I.
E-6	1	6.0	5.25	5.25	1-3/4	2.8	-	4240	740	800
	2	6.0	5.25	5.35	2	3.2	-	3325	615	685
	3	6.0	5.25	5.35	2-1/2	3.4	-	3730	550	780
	Ave.						4350*	3765	635	755
	Estimated for Center Loading								795	940
E-4	1	4.0	8.0	7.4	2-1/2	2.7	-	2335	420	575
	2	4.0	8.0	7.3	2	2.7	-	3085	515	700
	3	4.0	8.0	7.3	2	3.1	-	2835	460	650
	Ave.						3175*	2750	465	640
	Estimated for Center Loading								580	805
E-3.5	1	3.5	8.5	8.2	2	3.1	-	2220	410	625
	2	3.5	8.5	8.2	2	3.2	-	2250	410	475
	3	3.5	8.5	8.2	1-3/4	3.2	-	2545	385	625
	Ave.						2700*	2340	400	575
	Estimated for Center Loading								500	720

*Estimated

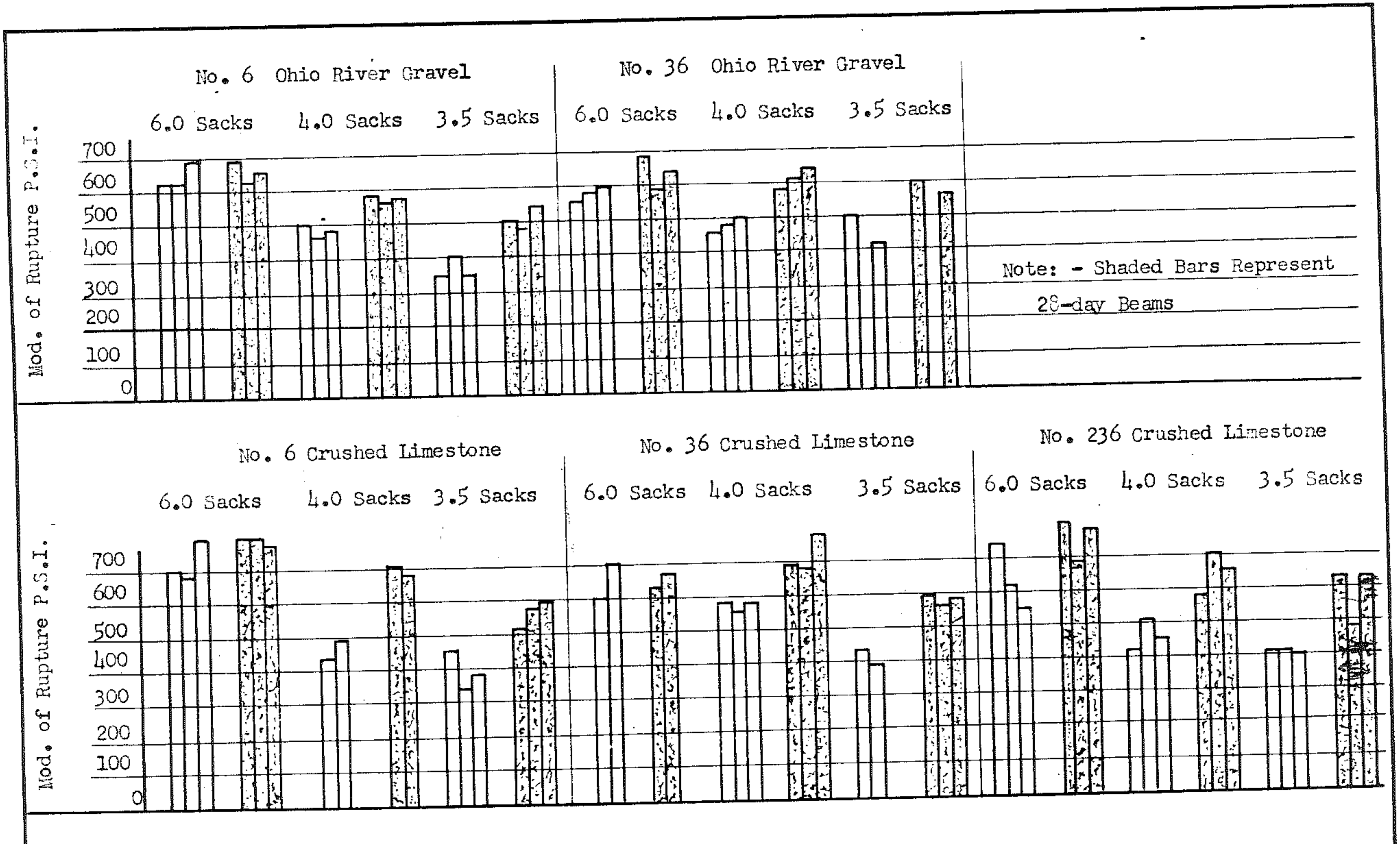


Fig. 10 - Moduli of Rupture of Individual 7-day and 28-day Laboratory Beams Tested Under Third-Point Loading.

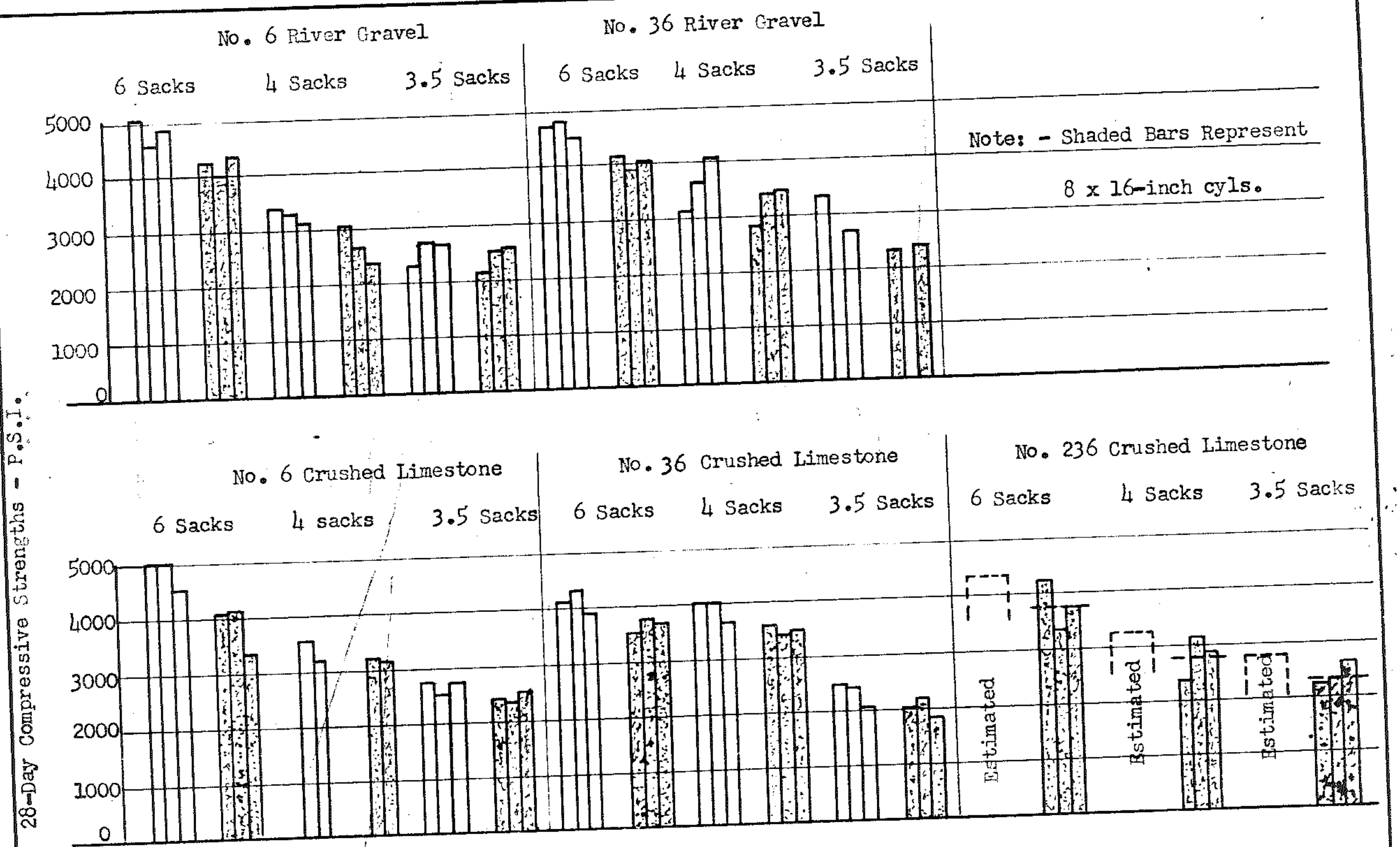


Fig. 11 - Compressive Strengths of Individual 6 x 12-inch and 8 x 16-inch Laboratory cylinders.

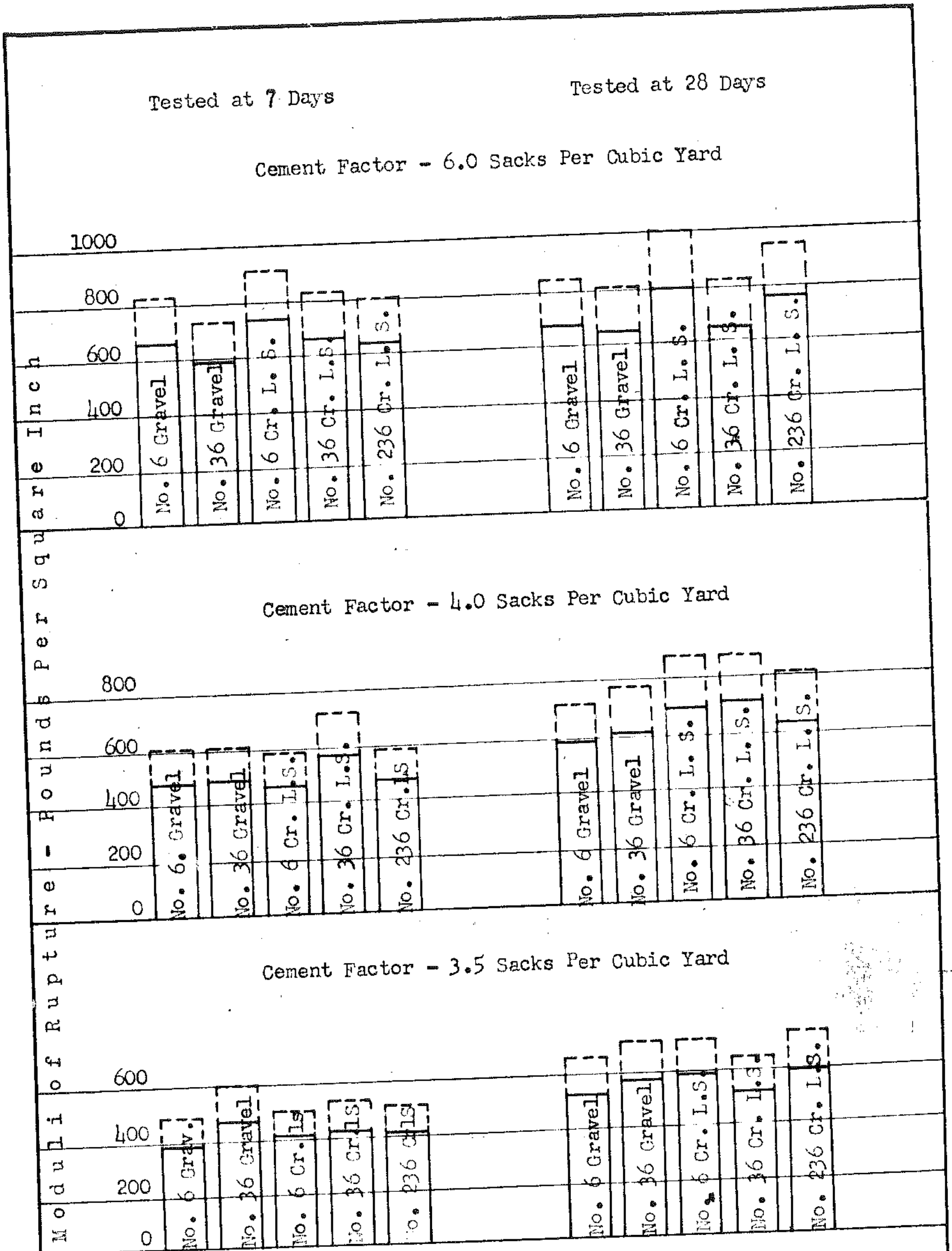


Fig. 12 - Averages of Moduli of Rupture for 7-day and 28-day beams. The solid Line bars represent actual values for third-point loading, and dashed extenions are estimated for center loading.

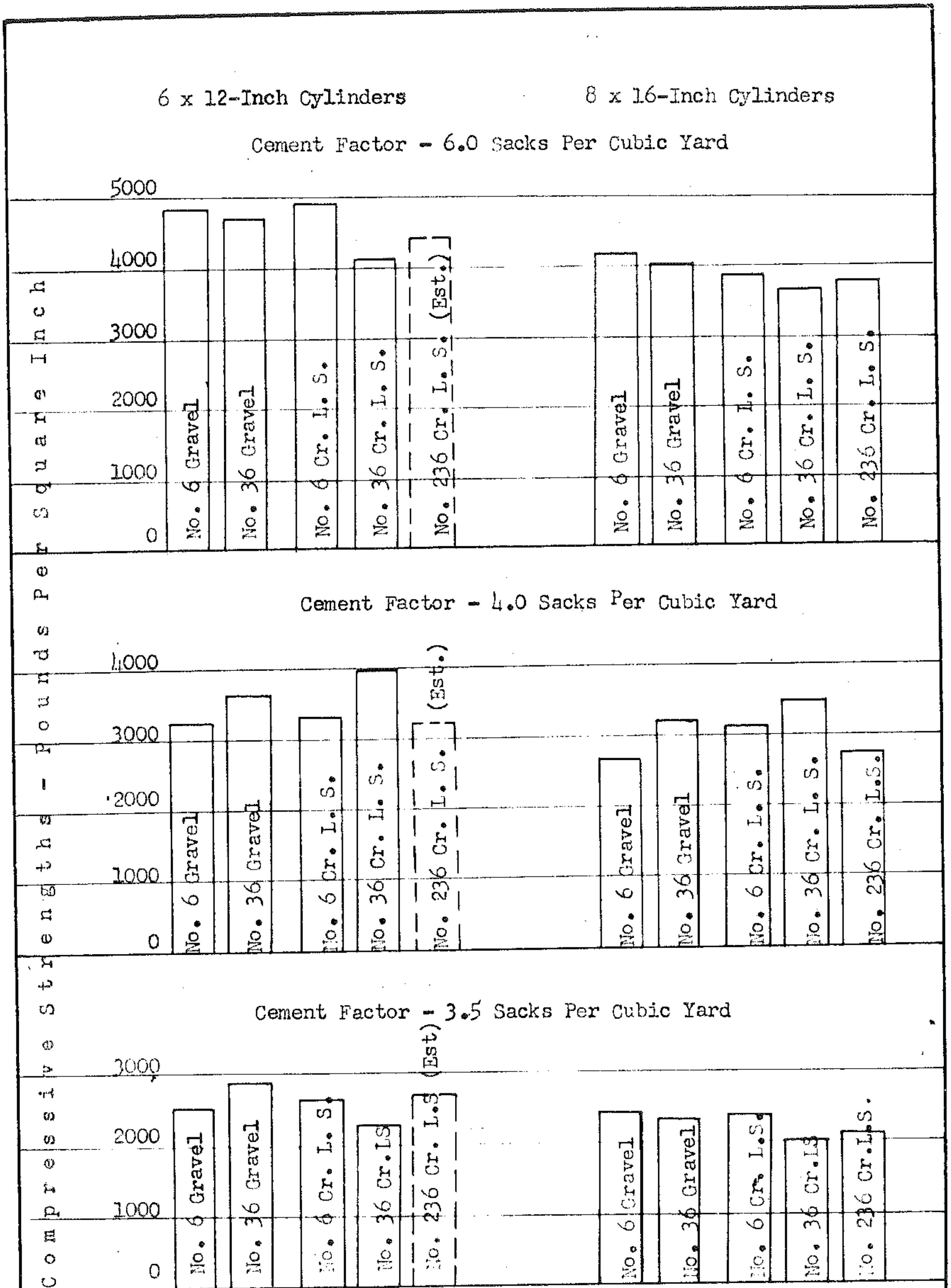


Fig. 13 - Compressive Strengths at 28-Days for 6 x 12-inch and 8 x 16-inch cylinders.

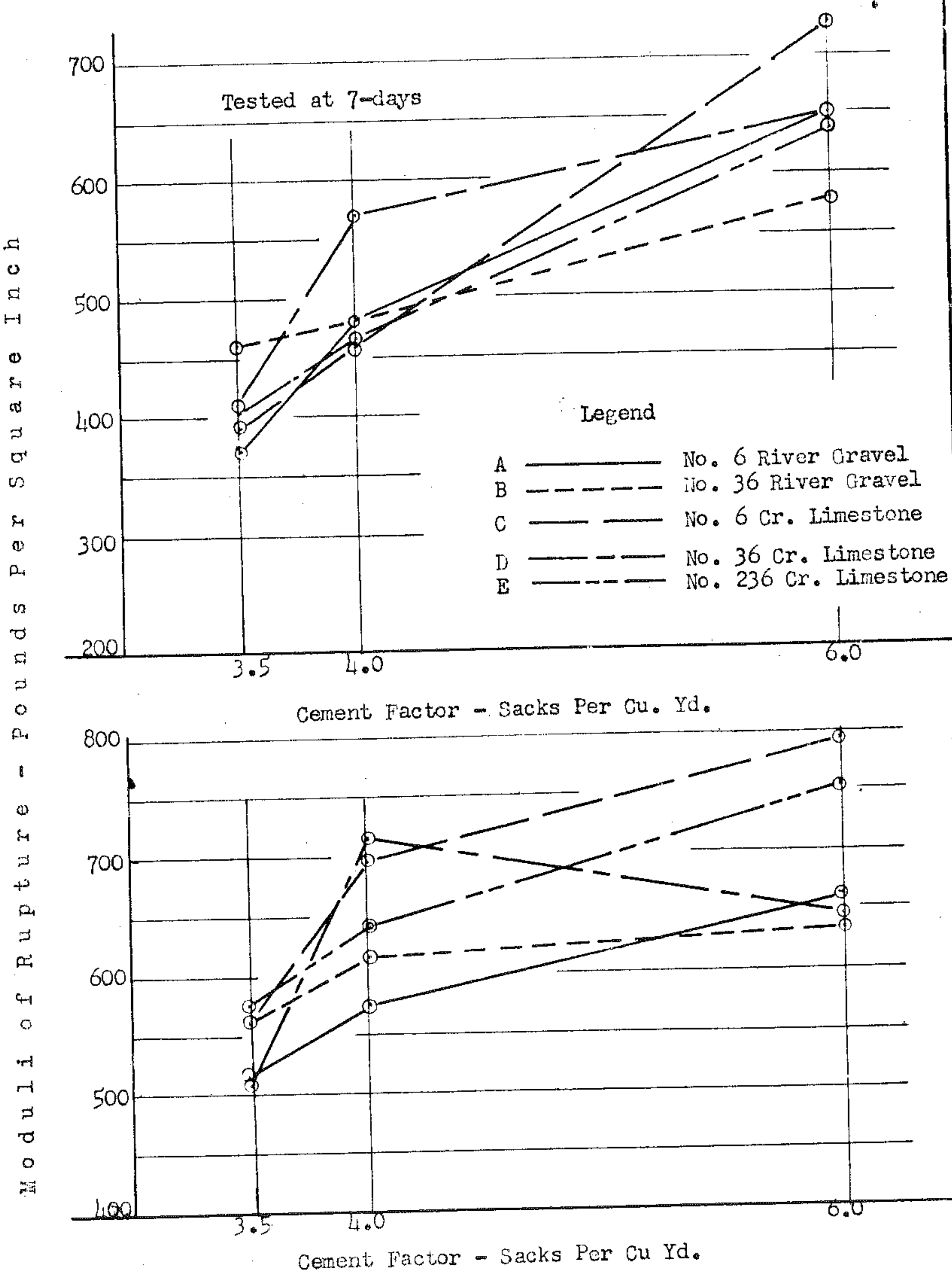


Fig. 14 - Strength-Cement Content Relation for 7-day and 28-day Laboratory Beams.

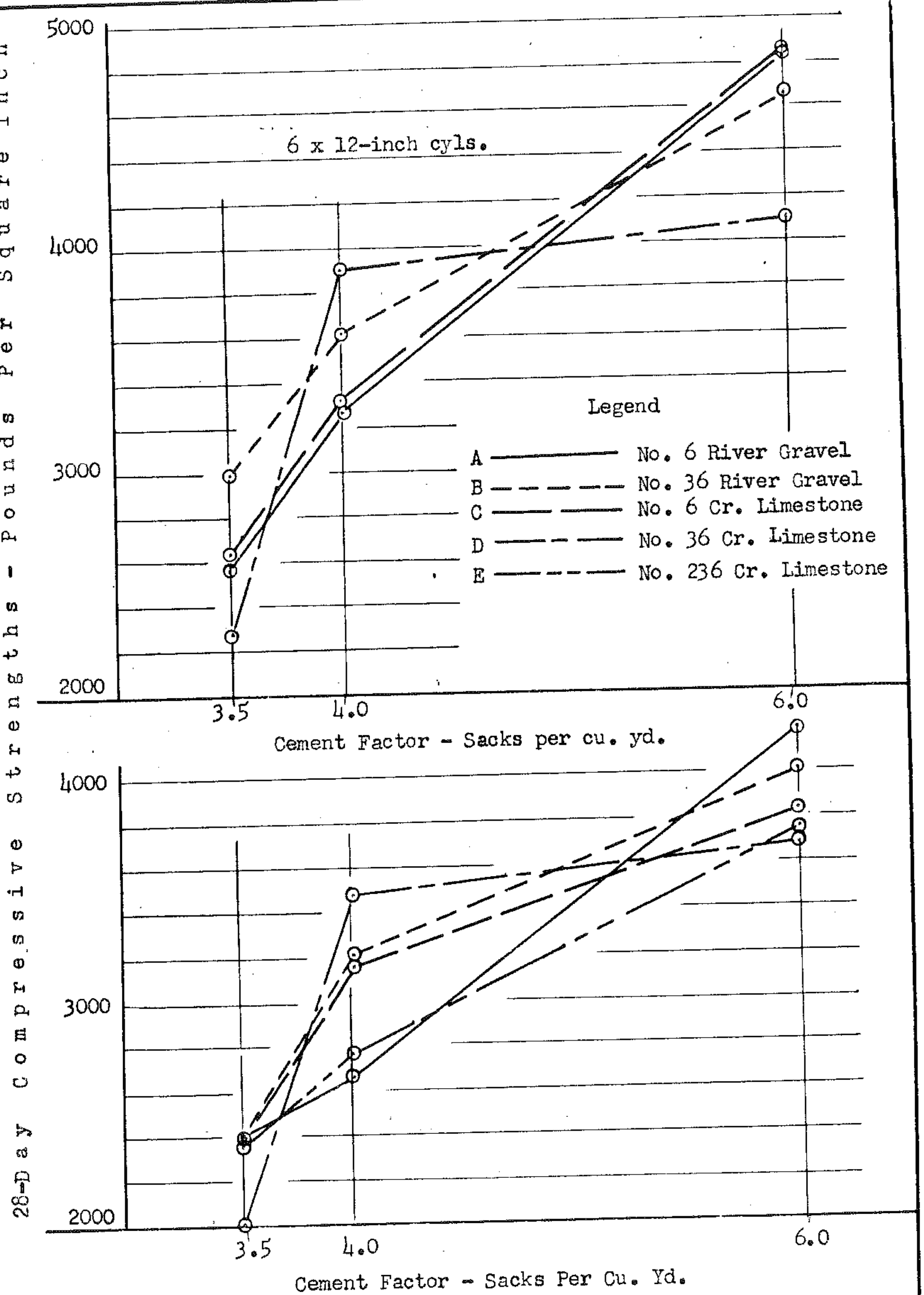


Fig. 15 - Strength-Cement Content Relations for 6 x 12-inch and 8 x 16-inch Concrete Cylinders.

TABLE VIII

CRACK FREQUENCY FOR BASE WIDENING

Left Side				Right Side			
	No.	No. Cracks			No.	No. Cracks	
Sta. to Sta.	Stas.	Total	Per Sta.	Sta. to Sta.	Stas.	Total	Per Sta.
0- 8	8	39	4.9	0- 9	9	17	1.9
8- 50	40.5	77	1.6	9- 26	17	2	0.1
				26- 28	2	11	5.5
				28- 34	6	1	0.2
50- 58	8	30	3.8	34- 39	5	16	3.2
59- 69	10	11	1.1	39- 72	28	12	4.4
69- 77	8	41	5.1	72- 84	12	12	1.0
77- 89	11	25	2.3	84- 89	5.5	31	5.6
				89- 97	7.5	2	0.3
89-109	20	209	10.5	97-105	8	79	9.9
109-114	4	7	1.8	105-114	8	7	0.9
124-134	10	57	5.7	124-134	10	13	1.3
142-148	6.5	41	6.3	142-148	6.5	13	2.0
155-159	4	25	8.2	155-162	7	18	2.6
159-165	6	12	2.0				
165-170	5	18	3.6				
170-177	7	11	1.6				
177-185	*	-	-	162-189	27	12	0.4
185-190	5	14	2.8				
196-218	21.5	31	1.4	196-227	31	9	0.3
218-230	12	49	4.1	227-231	3.5	12	2.9
230-236	6	6	1.0				
Totals	192.5	703			194	278	

A total of approximately 650 contraction joints are included in the right side.

*This section covered with wedge course before survey was made.

Full width sections are not included in this table.

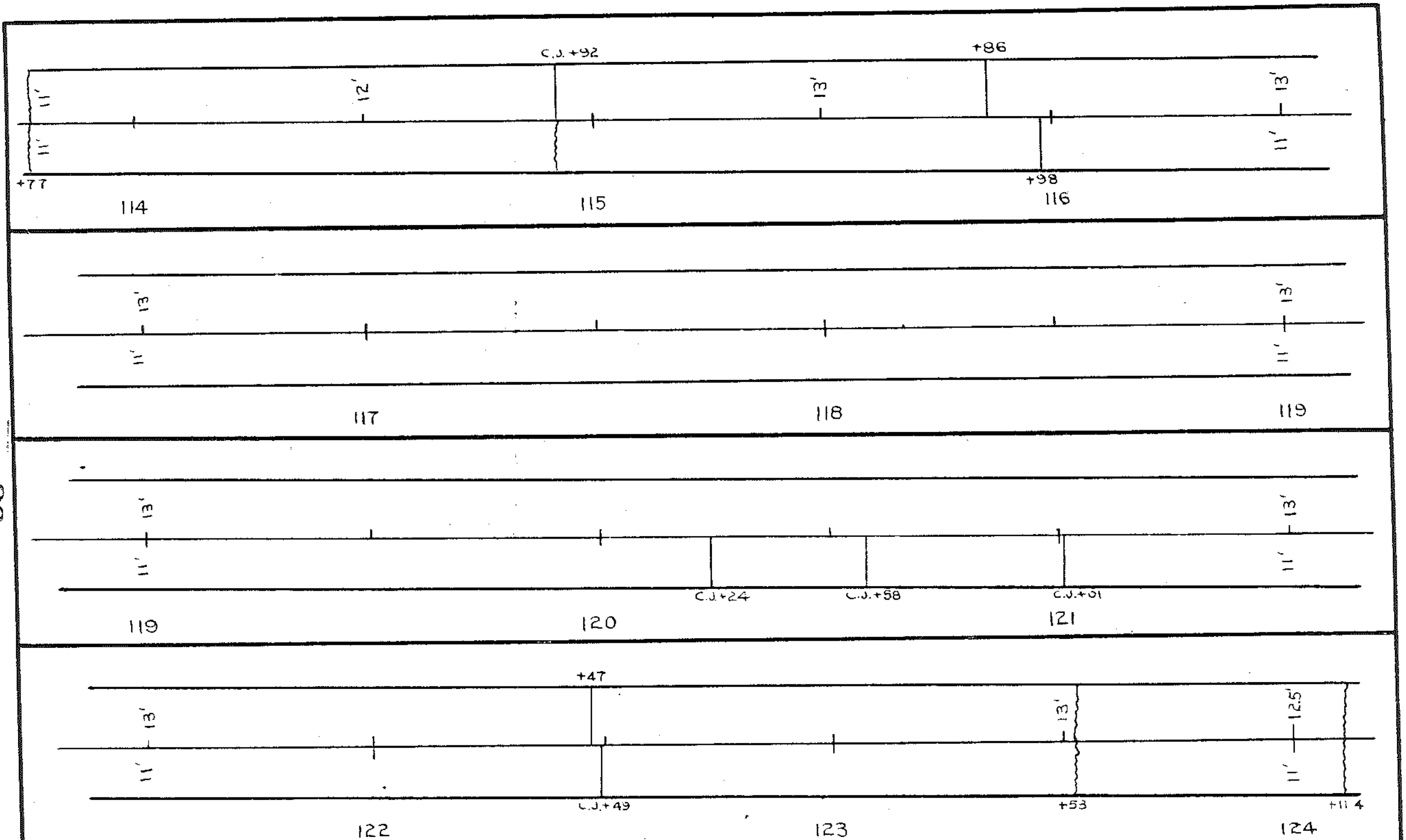


Fig. 16a - Layout of full width revision (Sta. 113 + 77 to Sta. 124 + 11) showing the location of cracks and joints.

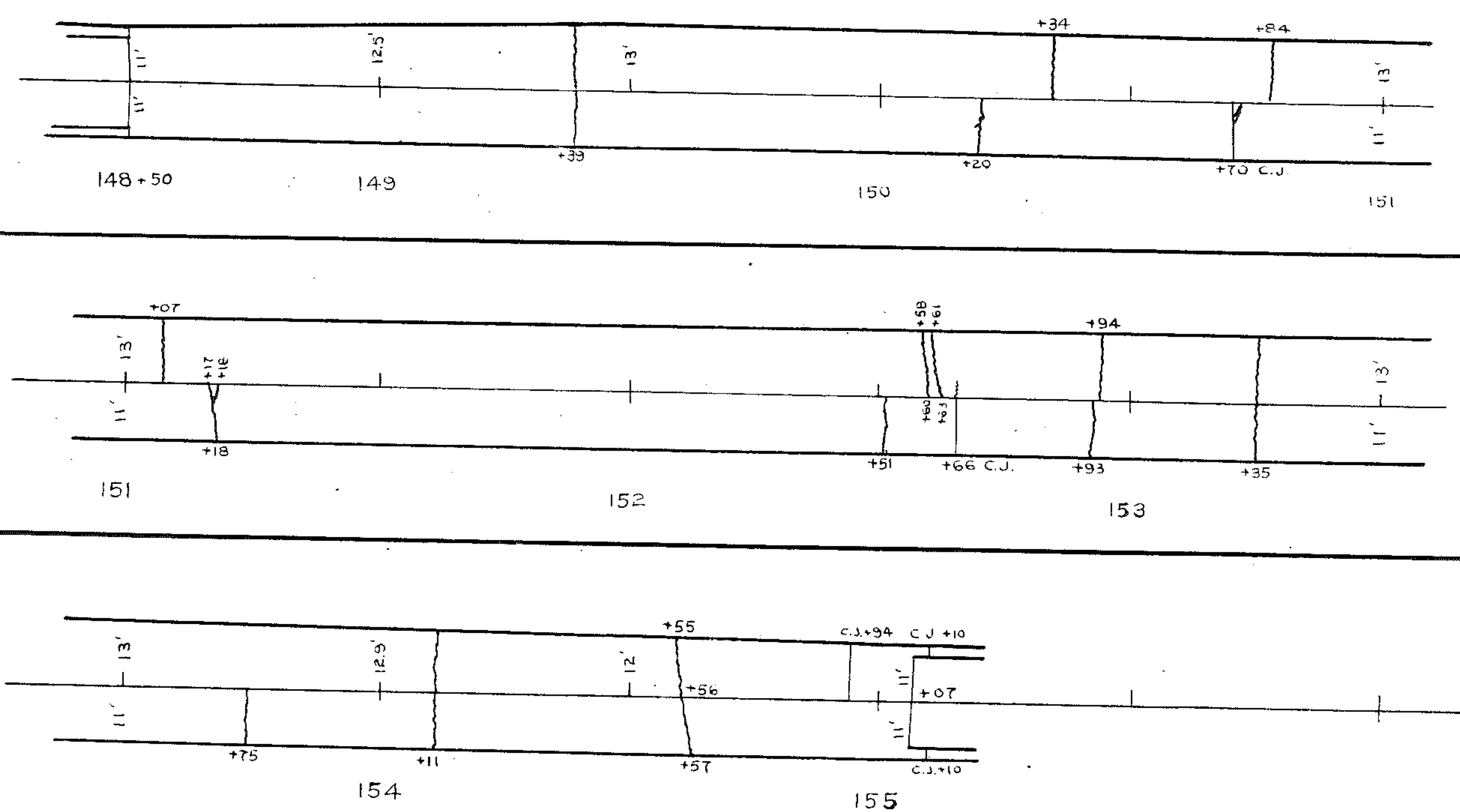


Fig. 16b - Layout of Full Width Revision (Sta. 148 ⨍ 50 to Sta. 155 ⨍ 07) Showing the Location of Cracks and Joints.

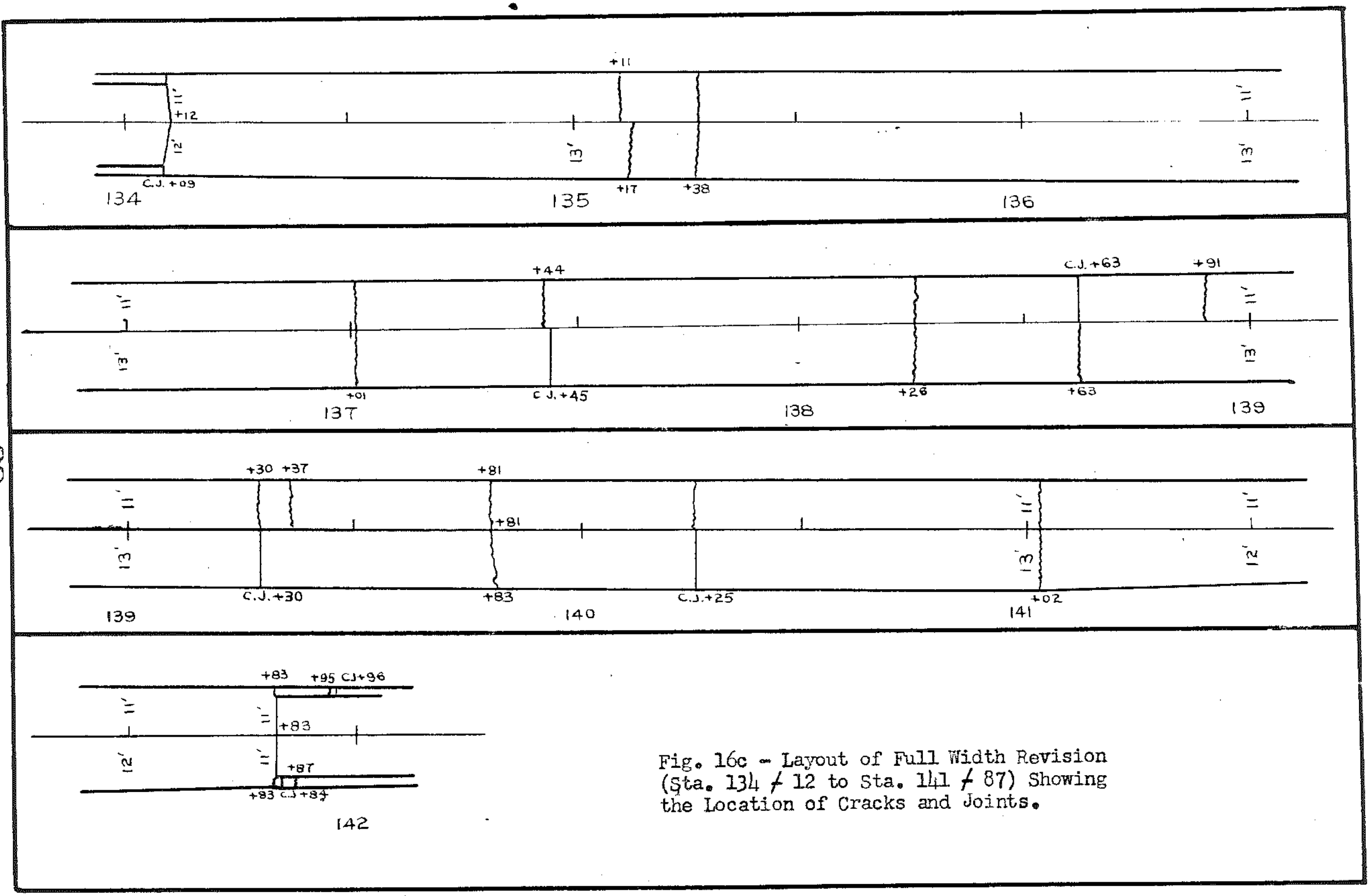


Fig. 16c - Layout of Full Width Revision (Sta. 134 + 12 to Sta. 141 + 87) Showing the Location of Cracks and Joints.

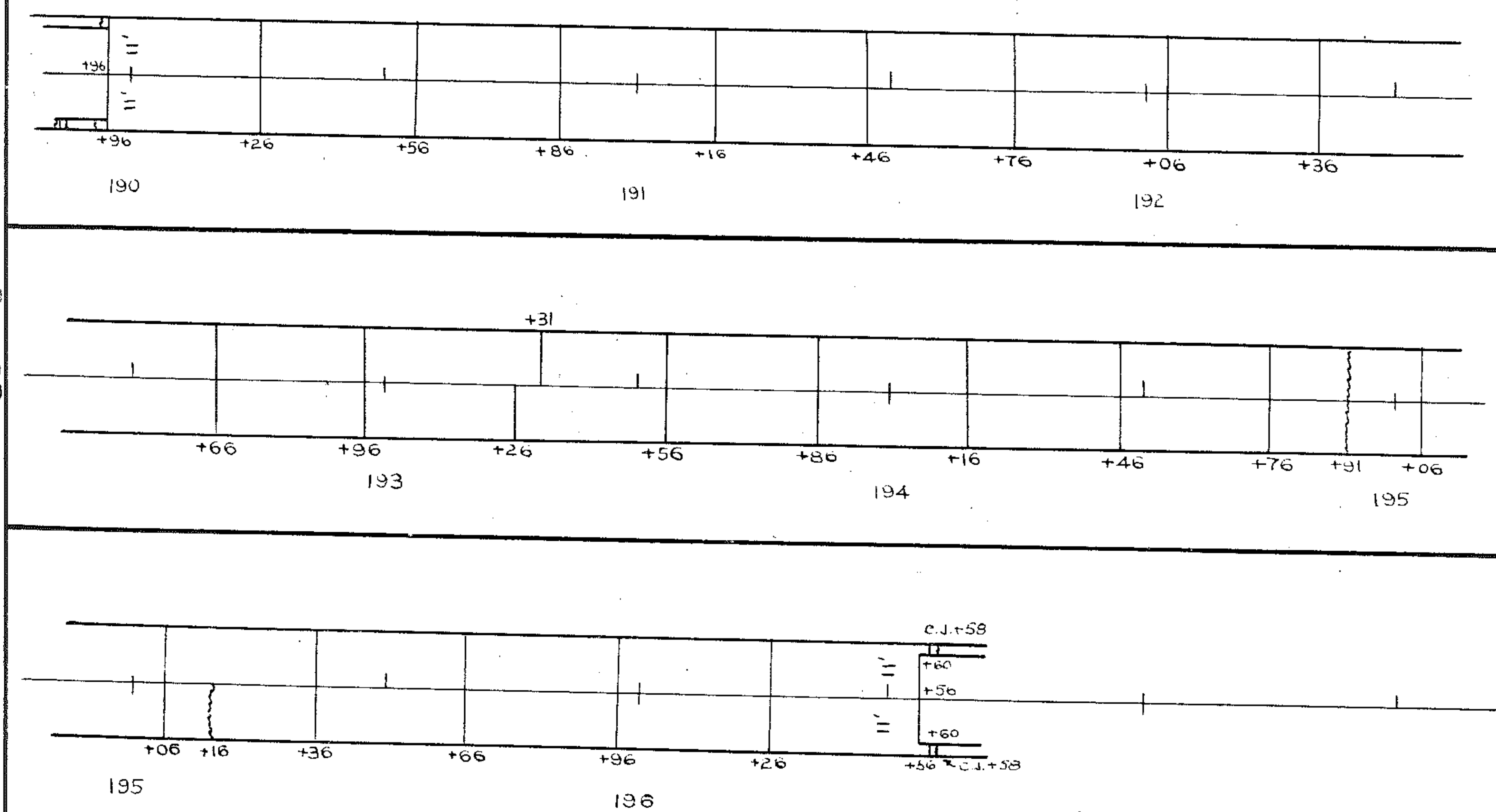
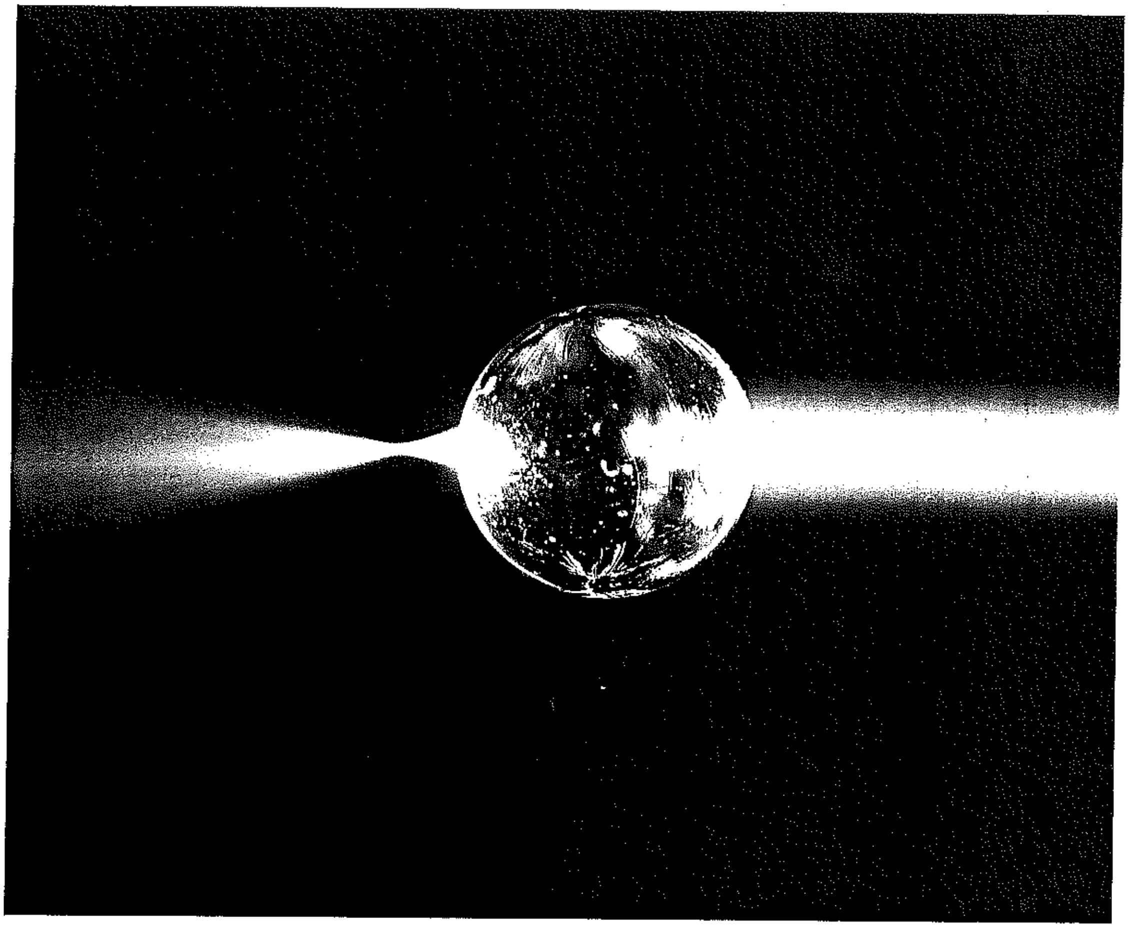


Fig. 16d - Layout of Full Width Revision (Sta. 189 + 96 to Sta. 196 + 56) Showing the Locations of Cracks and Joints.



Photograph showing the convergence of a beam of light by refraction at the surface of a glass sphere. This is a practical illustration of an otherwise theoretical analysis of the optical properties of sign surfaces.