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UNSTABLE SUBGRADE, I 65, HARDIN COUNTY (I 65-5(17)92; FSP 047-0065-091-094-0396)

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16. Abstract

During construction of the southbound lanes of I 65, Stations 120+00 to 218+00, in Hardin County, Kentucky, near Elizabethtown, large deformations were observed when dense-graded aggregate (DGA) base courses were loaded with construction traffic. Rutting and cracking of the DGA also was noticeable. A study was performed to determine the causes of the unstable subgrade and base, recommend remedial actions, and evaluate the effectiveness of the remedial measures. The stability, or bearing capacity, of the unstable pavement was determined for different construction stages and combinations of pavement layer thicknesses using a relatively new stability model HOPK-I. The stability of the pavement during early construction stages also was analyzed using Vesic's bearing capacity equations; results were compared to those obtained from the HOPK-I stability model. Initially, a geotextile fabric had been placed on the clay subgrade in an attempt to stabilize the subgrade. Remedial action consisted of mixing the top 6 to 9 inches of DGA with 7 to 10 percent of portland cement. A 3-inch layer of DGA was placed between the cement-DGA mixture and asphaltic concrete to minimize reflective cracking. Remedial actions were analyzed using the HOPK-I model and evaluated using the Road Rater. Based on stability analyses, the failure of the granular base was a result of a bearing capacity failure in the clay subgrade; the original design was inadequate. Stability analyses and Road Rater results showed that the remedial action vastly improved the structural behavior of the pavement. Results indicate that the HOPK-I model may be a good analysis tool. The study also emphasizes the need to determine and analyze the constructability of pavement designs.

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INTRODUCTION

In August 1984, subgrade and base problems developed during construction of the southbound lanes of a section of I 65 in Hardin County near Elizabethtown. The problem section is located between Stations 120+00 and 218+00 and is situated south of the junction of I 65 and the Bluegrass Parkway and north of the junction with the Western Kentucky Parkway (I-65-5(17)92; FSP 047-0065-091-094-0396). During construction of the dense-graded aggregate (DGA) base courses, large deformations were observed when the base courses were loaded with construction traffic. Rutting and cracking of the DGA course were also noticeable during construction before placement of the asphaltic concrete layer. Initially, the pavement design consisted of 16 inches of dense-graded aggregate and 7 inches of asphaltic concrete. A geotextile fabric also was used at the interface between the subgrade and DGA layer. Reportedly, a large portion of the new section of I 65 was placed over an original (old) section of I 65. The dense-graded aggregate was placed on a compacted 1-foot layer of silty clay. The upper foot of the old original subgrade was plowed, dried, and recompacted. In the latter part of 1984, an impromptu study of the unstable section was requested by the Kentucky Department of Highways.

OBJECTIVES AND SCOPE OF STUDY

The purposes of the study were to determine the causes of the unstable subgrade and base, recommend remedial action, and evaluate the effectiveness of the remedial measures. Remedial measures were evaluated using the Road Rater.

The stability, or bearing capacity, of the unstable pavement was determined at different construction stages using a relatively new slope stability model HOPK-I (1), which was partially adapted to analyze the stability of pavements. The stability of the pavement during early construction stages was also analyzed using Vesic's bearing capacity equations (2); the results were compared to results obtained from the HOPK-I stability model.

DATA COLLECTION, SAMPLING, AND TESTING

Efforts to determine the causes of the unstable condition consisted of reviewing available design and construction data, obtaining samples of the dense-graded aggregate, and testing the samples. Undisturbed Shelby tube samples of the subgrade were collected prior to this study from depths ranging from 1 to 3 feet by the Division of Materials. Unconfined compression tests and natural water content tests were performed on the tube samples. CBR tests, in-place water content tests, and dry densities of the subgrade materials were performed by personnel of the District Office at Elizabethtown. Bag samples of the dense-graded aggregate were obtained during construction and wet and dry sieve analyses were performed to determine the gradations.

TEST RESULTS AND ANALYSIS OF MATERIALS

A summary of in-place dry densities and water contents of the subgrade are shown in Table 1. Those tests results were obtained from the 1-foot compacted layer at the finished grade elevation before placement of the geotextile fabric and DGA. The upper foot of the subgrade was disked and allowed to dry to a water content near optimum (ASTM D 698). After drying, the material was recompacted. Relative density of this layer averaged 108 percent, ranging from 94.5 to 117.5 percent. Relative water contents (the ratio of in-place water content to optimum water content) ranged from 61.1 to 149.3 percent and averaged 94 percent. Hence, the in-place dry density was greater than the maximum dry density (ASTM D 698); the in-place water content was drier than optimum. This layer was initially compacted according to specifications.

Natural water contents of the original subgrade materials (before the top 1-foot was reworked and recompacted) are summarized in Table 2. CBR values also are shown in that table. Those data were obtained by district personnel. Optimum water contents (ASTM D 698) ranged from 18 to 24 percent. Natural (in-place) water contents ranged from about 23 to 30 percent. Relative water contents ranged from 97 to 152 percent and averaged 122 percent. Hence, the original subgrade material located below the reworked 1-foot compacted layer had water contents in excess of optimum.

In-place dry densities of the compacted dense-graded aggregate obtained during construction are shown in Table 3. Relative compaction of the different construction lifts of DGA ranged from 77.8 to 91.2 percent and averaged 85.7 percent. Specifications require a minimum relative compaction of 84 percent of solid density. The DGA layers appear to have been properly compacted.

Results from unconfined compression tests of the foundation soil are summarized in Table 4. Undisturbed Shelby tube samples for those tests were obtained from a depth ranging from 1 to 3 feet in the subgrade. The undrained shear strength ranged from 843 to 4,001 pounds per square foot and averaged 1,251 pounds per square foot. Natural water contents of the specimens ranged from 18.1 percent to 30.9 percent. The natural water content of the subgrade averaged about 24 percent. Generally,

TABLE 1. SUMMARY OF IN-PLACE DRY DENSITIES AND WATER CONTENTS OF THE TOP 1 FOOT OF COMPACTED SUBGRADE, I 65, SOUTHBOUND LANES, STATION 120+00 TO 218+00

STATION NUMBER	STANDARD COMPACTION (TARGET VALUES) OPTIMUM WATER CONTENT (%)	STANDARD COMPACTION (TARGET VALUES) DRY DENSITY (pcf)	IN-PLACE COMPACTION WATER CONTENT (%)	IN-PLACE COMPACTION DRY DENSITY (pcf)	IN-PLACE COMPACTION RELATIVE COMPACTION (%)	RELATIVE WATER CONTENT* (%)	DEPTH OF TEST (in.)
138+00	16.0	113.0	14.2	126.7	112.1	88.8	6
148+00	24.0	95.0	14.7	111.7	117.5	61.1	6
150+00	24.0	95.0	19.9	111.0	116.1	82.9	-
160+00	21.0	97.0	22.8	106.4	109.7	108.6	-
187+00	16.0	110.0	12.8	116.7	106.1	80.0	6
194+00	16.0	110.0	22.8	104.0	94.5	142.5	6
194+00 **	16.0	110.0	23.9	103.5	94.1	149.3	8
198+00	18.0	103.0	15.5	111.5	108.3	86.1	6
198+00	18.0	103.0	15.4	110.8	107.6	85.6	8
208+00	20.0	101.0	14.6	114.6	113.5	73.0	6
208+00	20.0	101.0	15.2	113.1	112.0	76.0	8

* The ratio of in-place water content to optimum water content.
** Retest.

TABLE 2. NATURAL WATER CONTENTS AND CBR'S SUBGRADE, SOUTHBOUND LANES, I 65, STATION 120+00 TO 218+00

STATION NUMBER	IN-PLACE WATER CONTENT (%)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	RELATIVE WATER CONTENT (%)	CBR
148+00	23.6	24.0	95.0	97.1	4.0
158+00	27.2	21.0	97.0	129.6	4.5
168+00	23.2	23.8	104.0	97.5	3.5
178+00	24.6	20.0	105.0	123.0	7.5
198+00	23.9	18.0	103.0	132.8	6.5
208+00	30.4	20.0	101.0	152.0	5.5

TABLE 3. SUMMARY OF DRY DENSITY TESTS PERFORMED ON THE DENSE-GRADED AGGREGATE (DGA), SOUTHBOUND LANES, I 65, STATION 120+00 TO 218+00

STATION NUMBER	OFFSET	THICKNESS OF LAYER (inches)	IN-PLACE COMPACTION: WATER CONTENT (%)	IN-PLACE COMPACTION: DRY DENSITY (pcf)	TARGET DRY DENSITY	RELATIVE COMPACTION (%)
FIRST LIFT						
144+50	Middle of Lane	8	6.1	142.0	159.1	89.2
152+00	Middle of Lane	8	5.1	143.9	159.1	90.4
156+00	Left Shoulder	8	5.2	145.2	159.1	91.2
156+00	Right Lane	8	6.2	142.8	159.1	89.7
162+00	Middle of Lane	8	3.9	130.3	159.1	81.8
162+00	Left Shoulder	8	4.6	141.6	159.1	89.2
163+50	Mainline	6	6.6	138.8	159.1	87.2
165+50	Mainline	6	4.2	132.5	159.1	83.3
165+50	Mainline	6	4.4	130.5	159.1	82.0
167+00	Mainline	6	6.4	142.8	159.1	89.7
SECOND LIFT						
140+00	Mainline	6	5.5	140.9	159.1	88.5
140+00	Mainline	6	5.5	142.3	159.1	89.4
166+50	Mainline	6	4.5	134.1	159.1	84.2
166+50	Mainline	6	4.8	133.6	159.1	83.9
169+00	Mainline	6	5.2	134.9	159.1	84.8
169+00	Mainline	6	5.2	136.7	159.1	85.9
171+00	Mainline	6	4.5	140.5	159.1	88.3
199+50	Mainline	6	5.2	136.9	159.1	86.0
202+00	Mainline	6	6.2	137.3	159.1	86.3
205+00	Mainline	6	5.6	138.9	159.1	87.3
205+50	Mainline	6	4.5	126.4	159.1	79.4
206+50	Mainline	6	6.3	136.5	159.1	85.8
GRADE ELEVATION						
134+00	Mainline	4	4.5	137.2	159.1	86.2
134+00	Mainline	4	3.8	134.5	159.1	84.5
134+00	Mainline	4	4.1	133.7	159.1	84.0
135+00	Mainline	4	4.5	139.7	159.1	87.8
140+50	Mainline	4	4.4	128.0	159.1	89.4
144+00	Mainline	4	4.7	133.8	159.1	84.1
144+00	Mainline	4	4.2	133.8	159.1	84.1
144+00	Mainline	4	4.8	123.9	159.1	77.8
148+50	Mainline, Middle Lane	4	3.6	131.6	159.1	82.7
154+00	Mainline	4	4.2	135.1	159.1	84.9
155+00	Mainline, Right Side	4	4.5	132.9	159.1	83.5
163+50	Mainline	4	5.2	139.7	159.1	87.8
170+50	Mainline	6	4.3	139.8	159.1	87.9

water contents were higher in certain portions of the unstable section than optimum water contents. As shown in Table 4, subgrade soils were clays and silty clays (CL). Shear strengths and CBR values of the subgrade were relatively low. CBR values ranged from 3.5 to 7.5 (Table 2).

Originally, a geotextile fabric was proposed to stabilize the subgrade. Based on results of unconfined compression tests and using methods of analyses contained in the FHWA Geotextile Engineering Manual (3), the required depth of granular material to be placed initially over the geotextile was determined (4). A summary of those results is presented in Table 5. It had been recommended that the geotextile fabric be placed over the unstable subgrade in accordance with Table III of Special Provision 39A (Kentucky Department of Highways). It also was recommended that light equipment be used to spread and compact 6 inches of DGA over the fabric. The recommendations also recognized that some rutting and pumping might occur under construction traffic after the 6 inches of DGA were placed over the fabric, since analyses indicated that 9 inches of DGA were needed in certain areas. It was recommended that heaved areas be covered with remaining lifts of DGA without prior grading or smoothing. Grading of the heaved areas would reduce the thickness of aggregate cover and promote more severe pumping. Also, it was recommended that hauling equipment for the additional DGA lifts should be routed over the entire width of roadbed. Reportedly, some rutting and deformations of the DGA occurred after the entire thickness of DGA (16 inches) had been placed.

Gradation analyses were performed on 12 bag samples of DGA. Both wet and dry sieve analyses were performed following ASTM D 2217 (also D

TABLE 4. UNCONFINED COMPRESSIVE STRENGTHS OF SUBGRADE SOILS, SOUTHBOUND LANES, I 65, STATION 120+00 TO 218+00

STATION NUMBER	OFFSET	VISUAL DESCRIPTION	UNDRAINED SHEAR STRENGTH (pcf)	DRY DENSITY (pcf)	WATER CONTENT (%)
139+50	45´Lt	Red Clay	4001	105.4	19.6
145+00	25´Lt	Red Clay	1030	88.6	30.4
150+00	51´Lt	Brown & Red Clay	1475	94.4	27.6
155+00	66´Lt	Brown & Gray Silty Clay	1364	94.0	19.3
160+00	63´Lt	Brown & Red Silty Clay	1033	105.4	18.1
	63´Lt	Brown & Red Silty Clay		105.7	
163+50	45´Lt	Brown Silty Loam (0 - 1.5´) Red Silty Loam (1.5´ - 3.0´)	857	97.2	23.9
171+50	66´Lt	Red Clay (0 - 1.5´) Brown Silty Loam (1.5´ - 3.0´)	1335	96.1	26.4
179+00	60´Lt	Red & Yellow Silty Clay	1611	89.7	30.9
	60´Lt	Red & Yellow Silty Clay		92.7	30.9
185+00	85´Lt	Red Silty Clay	1759	96.6	25.2
191+00	66´Lt	Red & Yellow Silty Clay	843	87.5	32.0
195+00	62´Lt	Brown Silty Loam	1004	101.5	20.9
202+50	63´Lt	Red & Yellow Silty Clay	1081	90.3	25.9

TABLE 5. INITIALLY REQUIRED THICKNESS OF AGGREGATE OVER GEOTEXTILE FABRIC

LOCATION		SAMPLE	UNCONFINED COMPRESSION (psf)	COHESION (psf)	AGGREGATE REQUIRED (inches)
139+50	45´Lt	1	8002	4001	0
145+00	25´Lt	1	2061	1030	8
150+00	51´Lt	1A	2950	1475	5
155+00	66´Lt	1	2727	1364	6
160+00	63´Lt	1A	2471		
		1B	1661	1033	8
163+50	45´Lt	1	2514	1257	6
171+50	66´Lt	1	2670	1335	6
179+00	60´Lt	1A	2223		
		1B	4220	1611	5
185+00	85´Lt	1A	3535		
		1B	3499	1759	4
191+00	66´Lt	1	1686	843	9
195+00	62´Lt	1	2008	1004	8
202+50	63´Lt	1	2161	1081	7

422) and D 421, respectively. Gradation curves are shown in APPENDIX A. Gradation limits, based on current specifications, also are shown in the 12 figures. Gradation of the DGA material is within specification limits except for the amount of material passing the No.-200 sieve. Comparisons of the percentage of material passing the No.-200 sieve and the specification limits are summarized in Table 6.

STABILITY ANALYSES

Stability, or bearing capacity, analyses were performed for various stages of construction and for different combinations of pavement layer thicknesses. The stability of the pavement was analyzed using a slope stability model and computer program (HOPK-I) (1). The mathematical model is a method of slices and is based on limited equilibrium concepts. The model essentially satisfies all three equilibrium equations and may be used to analyze shear surfaces of irregular shapes. This computer program has been partially adapted to analyze the stability of pavements. Initial construction of the granular (DGA) base course was analyzed using classical bearing capacity equations and a method described by Vesić (cf. 2). Vesic's equations considers a punching mode of failure. Failure is assumed to occur essentially along vertical slip lines. Generally, classical bearing capacity models consider only one-layered systems. In the method described by Vesic, a two-layered system may be analyzed. The HOPK-I stability model was used to analyze cases involving more than two layers. This method was also used to analyze early construction stages.

Dual truck wheel loading was represented in the bearing capacity analyses by a uniform, distributed load, as shown in Figure 1. The width of the dual wheels (and the uniform, distributed load) was assumed

TABLE 6. COMPARISON OF PERCENTAGES OF DENSE-GRADED AGGREGATE PASSING NO.-200 SIEVE AND SPECIFICATION LIMITS

SAMPLE NUMBER	PERCENTAGE OF DGA SAMPLE PASSING THE No.-200 SIEVE		SPECIFICATION LIMITS OF PERCENTAGES PASSING THE No.-200 SIEVE	
	DRY SIEVE	WET SIEVE	MAXIMUM	MINIMUM
8-29-O 1	16.0		12	5
8-28-M 2	15.0		12	5
8-28-P 3	15.0		12	5
4	16.2		12	5
5	16.0		12	5
8-29-Q 6	16.0		12	5
8-29-R 7	14.0		12	5
Sta 135+50	-	15.3	12	5
Sta 165+50	-	18.8	12	5
Sample A	-	15.0	12	5
Sample B	-	18.0	12	5
Sample C	-	7.5	12	5

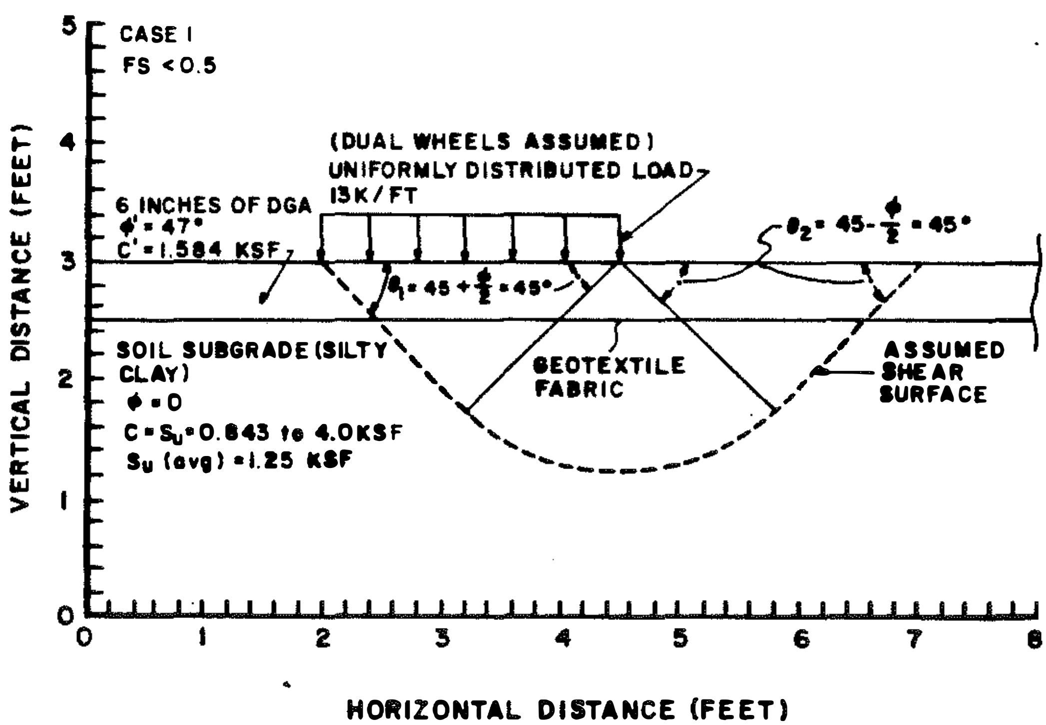

Figure 1. Assumed Failure Mode of a 6-inch Layer of Dense-Graded Aggregate on Subgrade, I 65, Southbound Lanes, Station 120+00 to 218+00.

to be 2.5 feet. The length (perpendicular to the width) was assumed to be 1 foot. Tire pressures of heavy, loaded trucks generally range from 80 to 100 pounds per square inch. Based on the assumed dimensions of the loaded area and a tire pressure of 90 pounds per square inch, a uniform distributed load of 13 kips per linear foot was calculated and used in the stability analyses.

Undrained shear strengths of the subgrade were obtained from unconfined compression tests. Those values are tabulated in Table 4. The undrained shear strengths ranged from 0.843 to 4.0 kips per square foot. However, the value of 4.0 kips per square foot occurred at only one location -- Station 139+50. The average undrained shear strength was 1.25 kips per square foot. Shear strengths of the bituminous concrete and dense-graded aggregate were obtained from a previous study (5). Values of the internal friction, ø, and cohesion, c, of the bituminous concrete were 37 degrees and 1.728 kips per square foot. Generally, the cohesive values of bituminous concrete range from 1.728 kips per square foot at high temperatures to 14.4 kips per square foot at low temperatures. The ø-value generally remained constant for extreme temperature ranges. The ø and c values for the DGA were 47 degrees and 1.584 kips per square foot (5).

Results obtained from the stability analyses are summarized in Table 7. Reportedly, rutting and cracking occurred during construction of the granular base. Two early construction stages were analyzed. Observations indicated that general shear failures and punching shear failures occurred during placement of the DGA. Case 1, as shown in Figure 1, considers the stability of a 6-inch layer of DGA on the subgrade subjected to a uniformily distributed loading of 13 kips per

TABLE 7. STABILITY (BEARING CAPACITY) ANALYSES OF PAVEMENT AT VARIOUS STAGES OF CONSTRUCTION AND DIFFERENT DESIGNS UNDER AN ASSUMED LOADING OF 90 POUNDS PER SQUARE INCH, SOUTHBOUND LANES, I 65, STATION 120+00 TO 218+00.

	SHEAR STRENGTHS OF PAVEMENT LAYERS													
	BITUMINOUS CONCRETE (7 In.)		DGA (3 In.)		CEMENT-DGA LAYER (6 In.)		DGA (2ND & 3RD LIFTS, 10 In.)		DGA (INITIAL LIFT, 6 In.)		SOIL SUBGRADE		FACTOR OF SAFETY (F)	BEARING CAPACITY (VESIC:2)
STAGE OF CONSTRUCTION	φ' (deg)	c' (ksf)	φ' (deg)	c' (ksf)	φ' (deg)	c' (ksf)	φ' (deg)	c' (ksf)	φ' (deg)	c' (ksf)	φ' (deg)	c' (ksf)		
1. 6 In. DGA	-	-	-	-	-	-	-	-	47	1.584	0.0	0.843	< 0.5*	**
	-	-	-	-	-	-	-	-	47	1.584	0.0	1.251	< 0.5*	**
	-	-	-	-	-	-	-	-	47	1.584	0.0	4.000	< 0.5*	0.13
2. 16 In. DGA	-	-	-	-	-	-	47	1.584	47	1.584	0.0	0.843	< 1.00*	0.88
	-	-	-	-	-	-	47	1.584	47	1.584	0.0	1.251	1.02-1.17	1.27
	-	-	-	-	-	-	47	1.584	47	1.584	0.0	4.000	1.83	4.23
3. 16 In. DGA (effective thickness reduced to 10-In.)	-	-	-	-	-	-	47	1.584	47	1.584	0.0	0.843	-	-
	-	-	-	-	-	-	47	1.584	47	1.584	0.0	1.251	0.82	0.75
	-	-	-	-	-	-	47	1.584	47	1.584	0.0	4.000	-	-
4. 6 In. DGA, 10 In. DGA	-	-	-	-	0.0	20.09	47	1.584	47	1.584	0.0	0.843	2.21	
	-	-	-	-	0.0	20.09	47	1.584	47	1.584	0.0	1.251	2.32	
	-	-	-	-	0.0	20.09	47	1.584	47	1.584	0.0	4.000	3.02	
5. 6 In. cement-DGA, 10 In. DGA reduced to 4 In. eff thickness	-	-	-	-	0.0	20.09	47	1.584	47	1.584	0.0	0.843	1.95	
	-	-	-	-	0.0	20.09	47	1.584	47	1.584	0.0	1.251	2.09	
	-	-	-	-	0.0	20.09	47	1.584	47	1.584	0.0	4.000	2.98	
6. As designed: 7 In. BC 16 In. DGA	37	1.728	-	-	-	-	47	1.584	47	1.584	0.0	0.843	1.66	
	37	1.728	-	-	-	-	47	1.584	47	1.584	0.0	1.251	1.72	
	37	1.728	-	-	-	-	47	1.584	47	1.584	0.0	4.000	2.15	
7. As designed: 7 In. BC 16 In. DGA reduced to 10 In. eff thickness	37	1.728	-	-	-	-	47	1.584	47	1.584	0.0	0.843	1.07	
	37	1.728	-	-	-	-	47	1.584	47	1.584	0.0	1.251	1.18	
	37	1.728	-	-	-	-	47	1.584	47	1.584	0.0	4.000	1.93	
8. As constr: 7 In. BC 3 In. DGA, 6 In. cement-DGA, 10 In. DGA	37	1.728	47	1.584	0.0	20.09	47	1.584	47	1.584	0.0	0.843	2.93	
	37	1.728	47	1.584	0.0	20.09	47	1.584	47	1.584	0.0	1.251	3.87	
	37	1.728	47	1.584	0.0	20.09	47	1.584	47	1.584	0.0	4.000	4.15	
9. As constr: 7 In. BC 3 In. DGA, 6 In. cement-DGA, 10 In. (Bot) DGA reduced to eff thickness of 4 In.	37	1.728	47	1.584	0.0	40.18	47	1.584	47	1.584	0.0	0.843	2.93	
	37	1.728	47	1.584	0.0	40.18	47	1.584	47	1.584	0.0	1.251	3.03	
	37	1.728	47	1.584	0.0	40.18	47	1.584	47	1.584	0.0	4.000	3.65	
10. 7 In. layer of BC and 16 In. of DGA of 12 In. of soil cement	37	1.728					47	1.584	47	1.584	0.0	0.843	4.04	
	37	1.728					47	1.584	47	1.584	0.0	1.251	4.04	
	37	1.728					47	1.584	47	1.584	0.0	4.000	4.04	
11. 7 In. layer of BC and 16 In. of DGA on soil subgrade***	37	1.728					47	1.584	47	1.584	0.0	0.843	1.38 (HDGA = 60.0 ft)	
	37	1.728					47	1.584	47	1.584	0.0	0.843	1.48 (HDGA = 37.5 ft)	
	37	1.728					47	1.584	47	1.584	0.0	0.843	1.56 (HDGA = 20.8 ft)	
	37	1.728					47	1.584	47	1.584	0.0	0.843	1.66 (HDGA = 0.0 ft)	

* Factor of Safety too low - computer program did not converge to a solution.

** Solution could not be obtained.

*** Undrained Shear Strength of soil-cement equal to 17.568 ksf (6). Effective stress analysis performed. Various pore pressures assumed in DGA Layer. HDGA equal excess pressure head of water acting in DGA layer.

linear foot. Using undrained shear strengths of 0.843, 1.251, and 4.0 kips per square foot for the subgrade soils, bearing capacity analyses yielded factors of safety of less than 0.5. The stability program also indicated factors of safety of less than 0.5. Because the factors of safety were so small, the computer program would not converge to a fixed solution.

Case 2 in Table 7 considers the stability of a 16-inch layer of DGA on the subgrade. Based on bearing capacity analyses, factors of safety of 0.88, 1.26, and 4.23 were obtained. Those values correspond to undrained shear strengths of the subgrade of 0.843, 1.25, and 4.0 kips per square foot, respectively. Analyses of the stability of the 16-inch DGA layer using the HOPK-I program yielded factors of safety of 0.84, 1.02, and 1.83. Factors of safety of 0.88, 1.27, and 4.23 were obtained from Vesić equations (2). These values corresponded to undrained shear strengths of 0.843, 1.25, and 4.00 kips per square foot, respectively. The failure surface assumed for bearing analyses using the computer program is shown in Figure 2. Based on the results of Cases 1 and 2, the cracking and rutting of the DGA was a result of a bearing capacity failure of the soft clays of the subgrade. Shear strengths of the subgrade (clays) were too low to support heavy construction equipment. Also, measurements of in situ water contents of the subgrade showed that the subgrade clays were too wet. High values of in situ water contents also indicate low shear strengths.

Case 3 assumes a 16-inch layer of DGA is effectively reduced to a thickness of 10 inches. This situation could develop if failures occurred during construction of the first lift of DGA. A factor of safety of 0.82, corresponding to an average undrained shear strength of

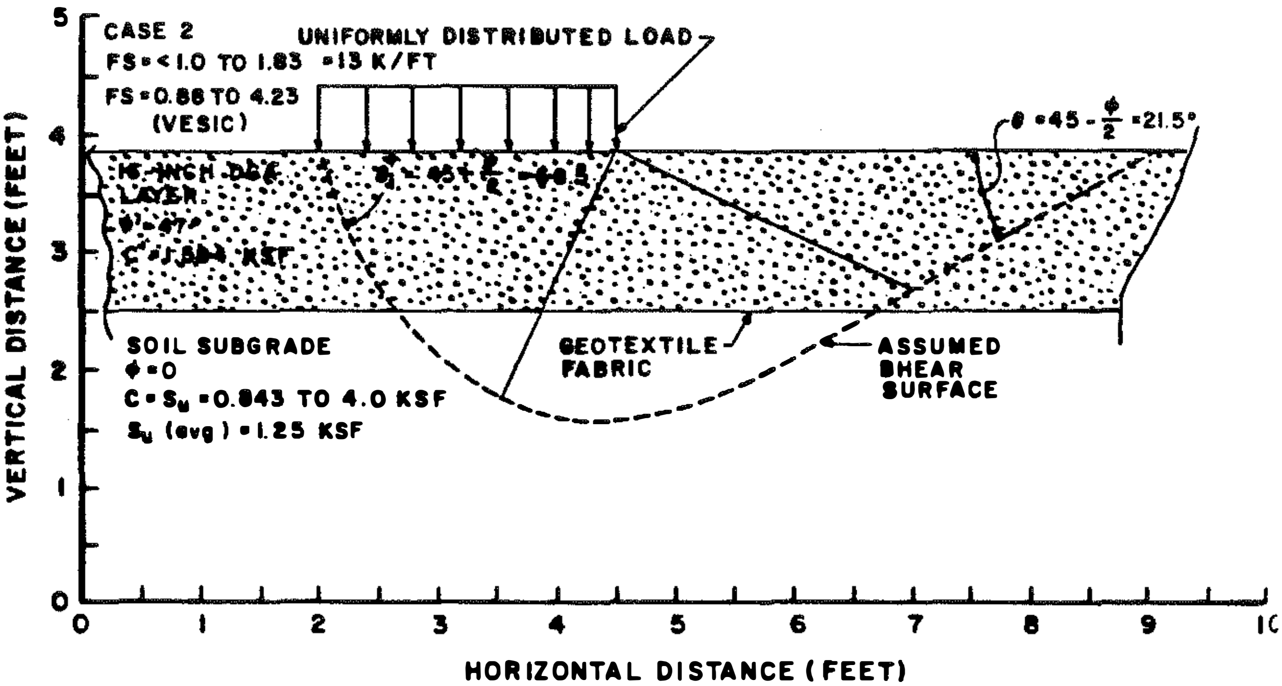

Figure 2. Assumed Failure Mode of a 16-inch Layer of Dense-Graded Aggregate on Subgrade, I 65, Southbound Lanes, Station 120+00 to 218+00.

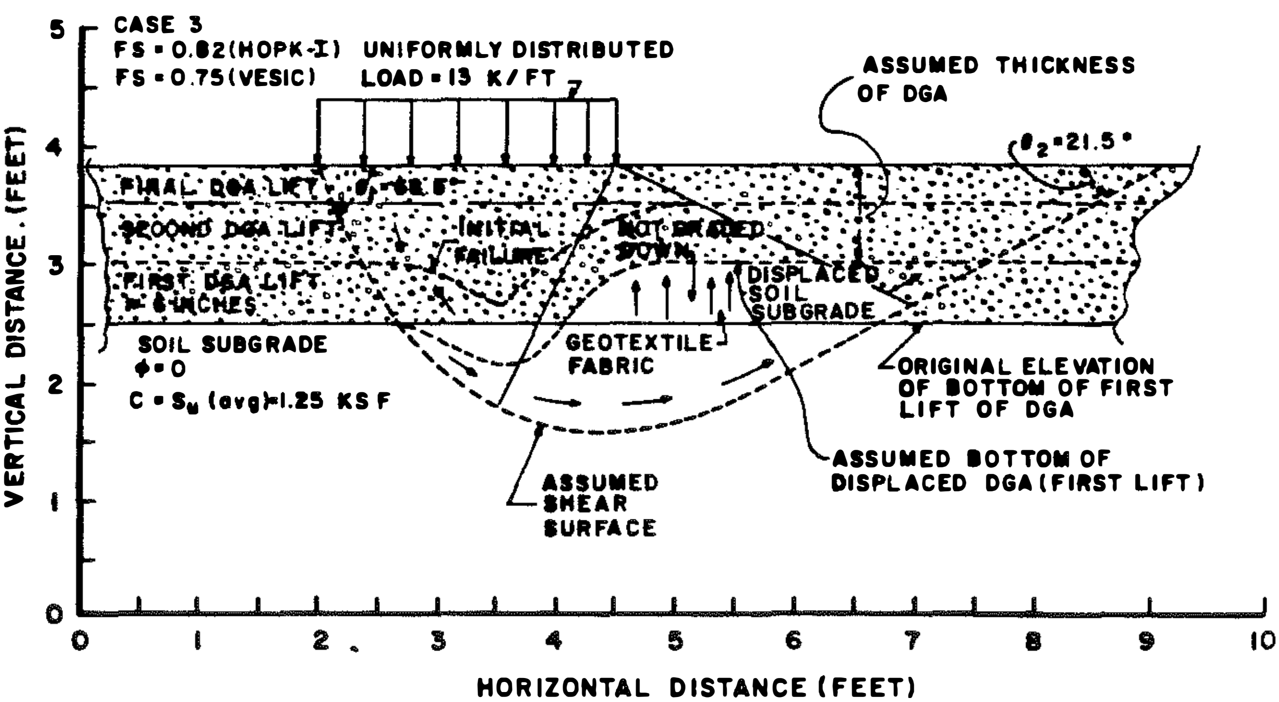

Figure 3. Assumed Failure Mode of a Reduced Section of Dense-Graded Aggregate, I 65, Southbound Lanes, Station 120+00 to 218+00.

1.25 kips per square foot, was obtained. Vesic's equations yielded a factor of safety of 0.75. Hence, these analyses indicate failure would occur during construction.

The original pavement design of the I-65 southbound lanes consisted of 7 inches of bituminous concrete on 16 inches of DGA (Figure 4). A geotextile (filter) fabric was to be placed on the soil subgrade. The bituminous concrete and DGA were to be placed on the fabric. To determine the adequacy of the initial design with regard to a bearing capacity failure, the stability of the original design was determined using the HOPK-I stability computer program. Results of those analyses are shown in Table 7 as Case 4. Factors of safety were 1.66, 1.72, and 2.15, which correspond to undrained shear strengths of 0.084, 1.25, and 4.00 kips per square foot, respectively. Appropriate factors of safety have not yet been established for analyses such as presented herein. Normally, factors of safety of about 2.5 or 3.0 are used for permanent structures. Excess pore pressures in the DGA layer were assumed equal to zero in those analyses. However, if the DGA was saturated, then large excess pore pressures could occur and would lower the factor of safety; that is, the factor of safety would be lower than the 1.66 obtained from the total stress analysis.

Case 4 analyses indicate the original design might be adequate. However, the design is based on the premise that the composite pavement thickness of 23 inches may be placed without failure. This premise is valid only when a general bearing capacity failure of the DGA-subgrade system does not occur under construction loadings. As shown by results of Cases 1 and 2, the DGA-subgrade would not support heavy construction equipment. Based on visual observations, the DGA-subgrade section most likely failed as illustrated in Figure 3. The fabric prevented the intrusion of the clay subgrade into the DGA. Hence, "mudwaves" and a general shear failure occurred as shown in Figure 3. Since no grading was performed in placing the lifts of DGA, the pavement thickness was likely reduced in areas not under the wheel loads of construction equipment.

In rutted sections, the pavement thickness is likely greater than the design thickness while in sections outside the rutted areas the actual thickness is probably thinner. Hence, the design thickness of asphaltic concrete is variable and may be effectively reduced in some areas. Case 5 (Figure 5) in Table 7 considers this situation. In those analyses, the 16-inch layer of DGA is assumed to be effectively reduced to 10 inches. The assumed composite pavement section -- 7-inch layer of bituminous concrete and 10-inch layer of DGA -- was analyzed and yielded factors of safety of 1.07, 1.18, and 1.83. Those values correspond to subgrade undrained shear strengths of 0.843, 1.25, and 4.00 kips per square foot, respectively. The low factors of safety strongly indicate that, if the pavement were constructed "as designed" and "as initially constructed", then imminent pavement failure would be likely when placed in service because of inadequate bearing capacity. Additionally, the apparent lack of foundation support would likely decrease the fatigue life of the section if failure was not immediate. Based on an interview with the resident engineer, the situation illustrated in Figure 3 apparently occurred. According to the resident engineer, when the upper 6 to 9 inches of the 16-inch layer of DGA was plowed to mix in portland cement, the disk of the plow "cut" the fabric at several locations. Such a situation apparently could occur only when the subgrade had

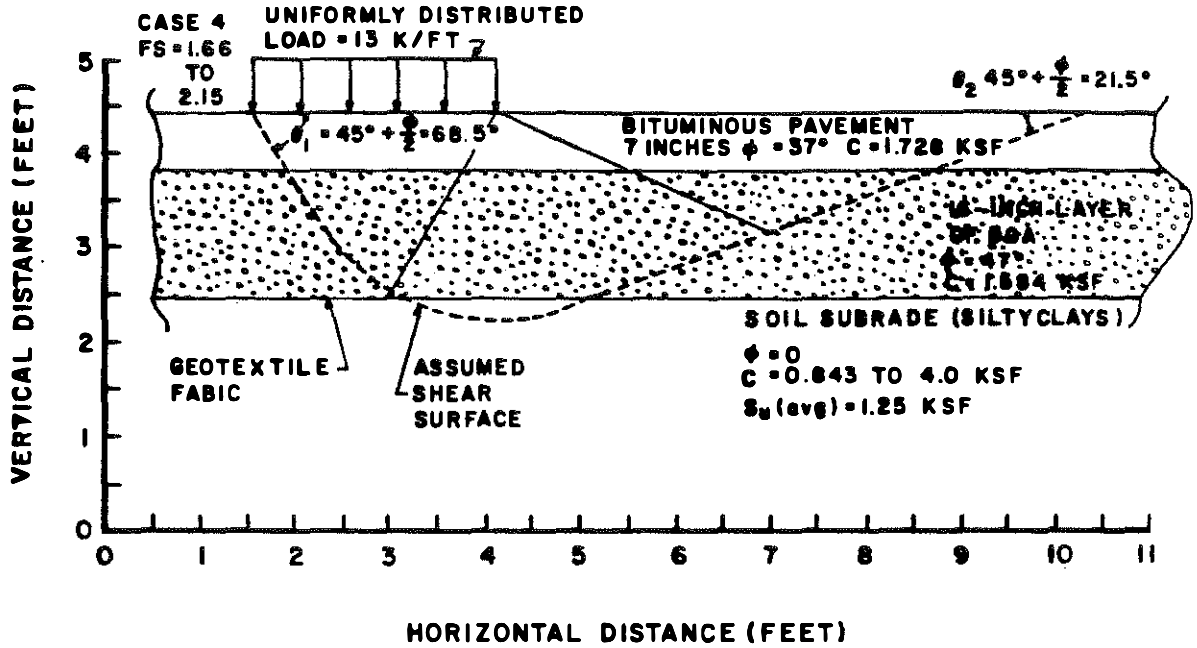

Figure 4. Assumed Failure Mode of the Original Pavement Design, I 65, Southbound Lanes, Station 120+00 to 218+00.

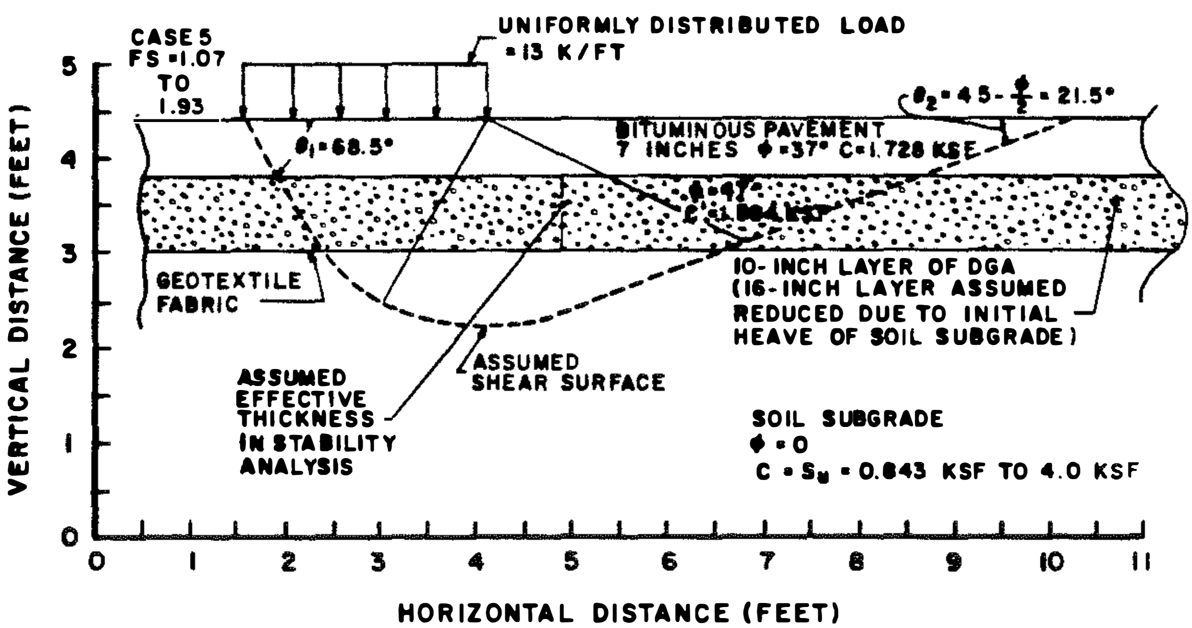

Figure 5. Assumed Failure Mode of the Original Pavement Design; 16-Inch Layer of Dense-Graded Aggregate Assumed Reduced to 10 Inches due to Initial Heave of Subgrade, I 65, Southbound Lanes, Station 120+00 to 218+00.

heaved as illustrated in Figure 3. Hence, remedial actions, as described below, were essential at this site to prevent premature failure.

The remedial action consisted of mixing the top 6 to 9 inches of DGA with approximately 7 to 10 percent portland cement. Four cylindrical specimens of that mixture were obtained from Stations 127+00 and 131+50 during the mixing operation. Those specimens were allowed to "cure" in the laboratory. After curing for 7 and 14 days, unconfined compression tests were performed. Results of those tests are shown in Figures B-1, B-2, B-3, and B-4 of APPENDIX B. The 7-day strengths ranged from 32 to 80 kips per square foot. The 14-day strengths ranged from 21 to 126 kips per square foot. In the remedial plan, a 3-inch layer of DGA was placed on the treated 6-inch layer of cement-treated DGA. In the event the cement-treated DGA layer cracks, the 3-inch layer of DGA would (perhaps) prevent reflective cracking of the bituminous concrete pavement.

Stability analyses of the pavement after repairs are illustrated by Cases 6 through 9 in Table 7. The undrained shear strength of the 6-inch cement-treated DGA layer was assumed equal to 20.9 kips per square foot (the lower value of the 14-day strengths in APPENDIX B). That value was treated as the undrained shear strength, $S_u = c$, in the analyses. The angle of friction was assumed equal to zero for the cement-treated DGA layer. Case 6 considers the 6-inch layer of cement and DGA mixture and the 10-inch layer (Figure 6). Factors of safety of 2.21, 2.32, and 3.02 corresponding to undrained shear strengths of the subgrade of 0.843, 1.25, and 4.00 kips per square foot, respectively, were obtained. Case 7 considers 6 inches of cement-treated DGA and 4 inches of DGA (Figure 7). The 10 inches of DGA was effectively reduced to 4 inches as shown in Figure 3. Factors of safety of 1.95, 2.09, and 2.98 were obtained. Cases 8 and 9 consider the "as constructed" pavement after remedial measures were implemented. Assumed shear surfaces are shown in Figures 8 and 9, respectively. In Case 8, the factors of safety were 3.83, 3.87, and 4.15, respectively. In Case 9, which considers a reduced composite thickness, factors of safety of 2.93, 3.03, and 3.65 were obtained. Those analyses strongly indicate that remedial measures were adequate to prevent premature failure of the DGA base and asphaltic concrete pavement during and after construction.

Soil-cement stabilization has been used on other projects in the area. In that method, the top foot of the subgrade is mixed with 7 to 10 percent portland cement. Assuming an undrained shear strength of 17.6 kips per square foot (6) for the top foot of the cemented-treated soil, the stability of 16 inches of DGA resting on the soil-cement, as shown in Figure 10, was analyzed. A factor of safety of 4.04 was obtained. Using different values of undrained shear strength for the untreated subgrade did not affect the factor of safety because the shear surface is located above the untreated subgrade and passes through the soil-cement layer as shown in Figure 10. This technique appears to yield a very stable pavement.

In the stability analyses described (Cases 1 through 10), pore pressures in the DGA layer were assumed equal to zero. The stability of the pavement was investigated using a total stress analyses, although drained shear strength parameters, ϕ' and c', were used for the DGA layer. The drained parameters could be used since the pore pressures in the DGA were assumed equal to zero. Undrained shear strength data were

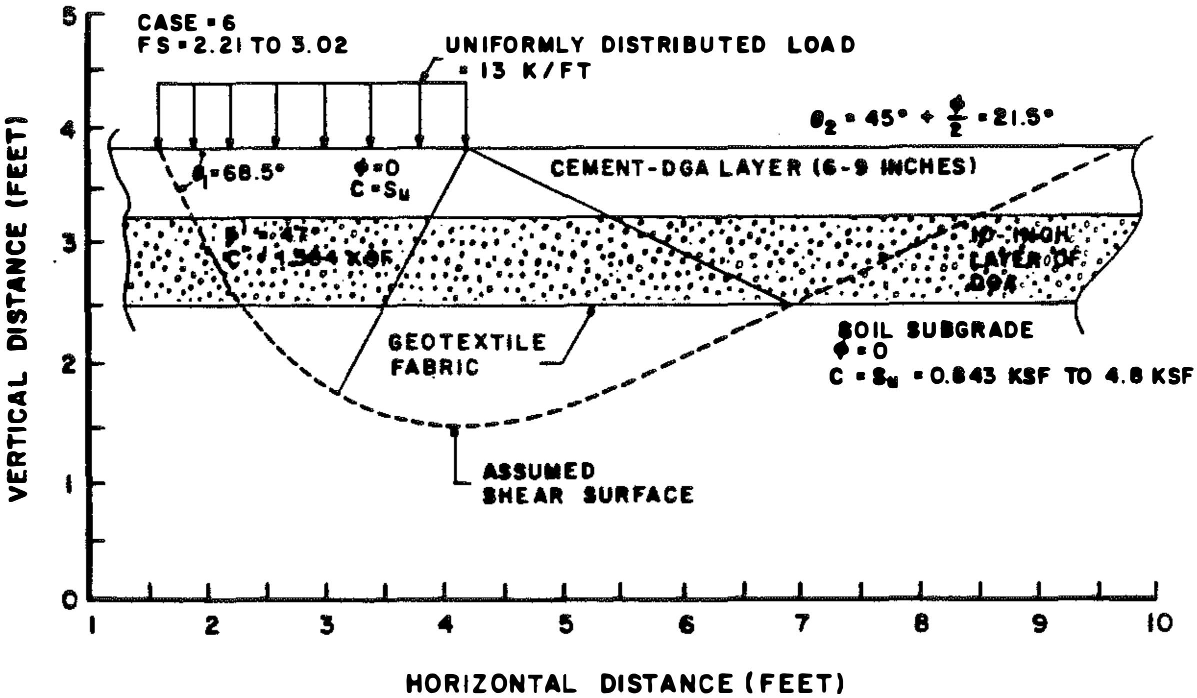

Figure 6. Assumed Failure Mode of a 6-inch Layer of Cement-Dense-Graded Aggregate Mixture and a 10-inch Layer of Dense-Graded Aggregate on Subgrade.

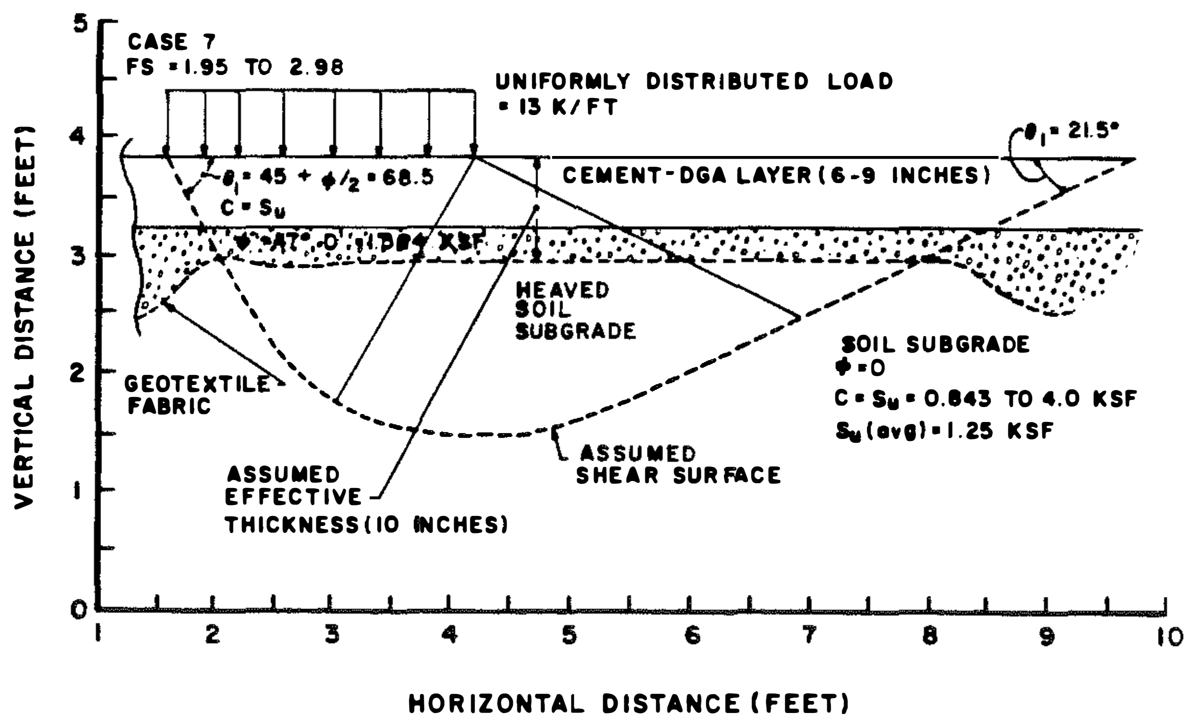

Figure 7. Assumed Failure Mode of a 6-inch Layer of Cement-Dense-Graded Aggregate Mixture and an Assumed Reduced Layer of Dense-Graded Aggregate due to Heaved Subgrade, I 65, Station 120+00 to 218+00.

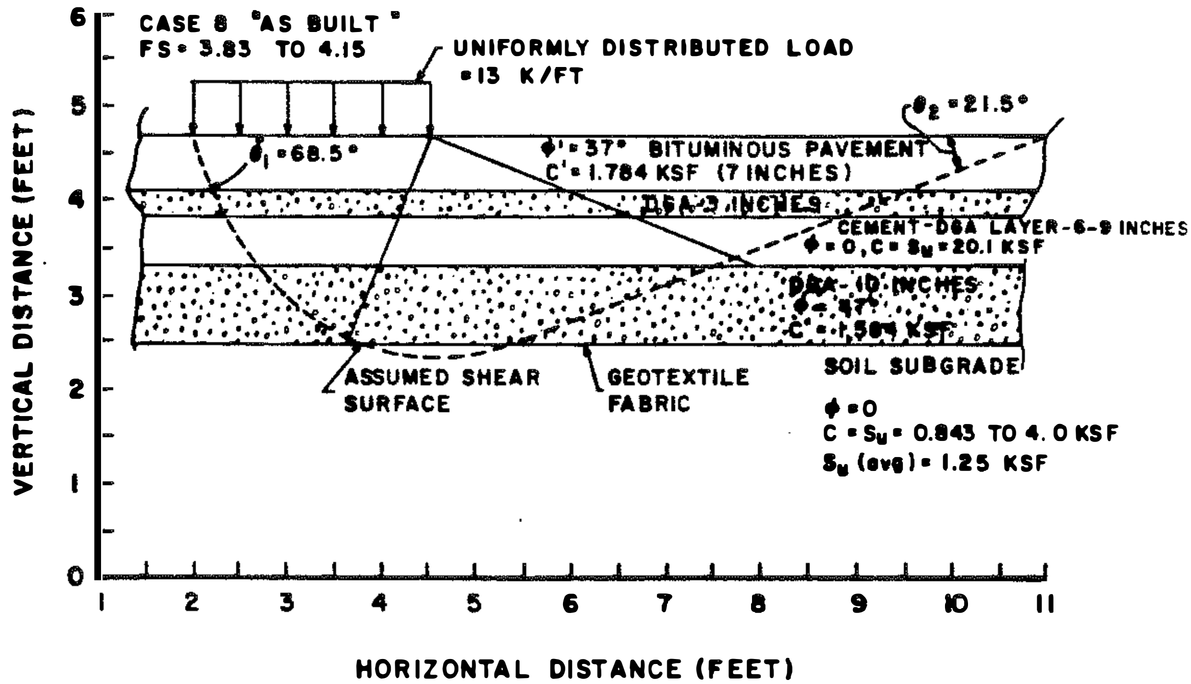

Figure 8. Assumed Failure Mode of the Remedial Pavement.

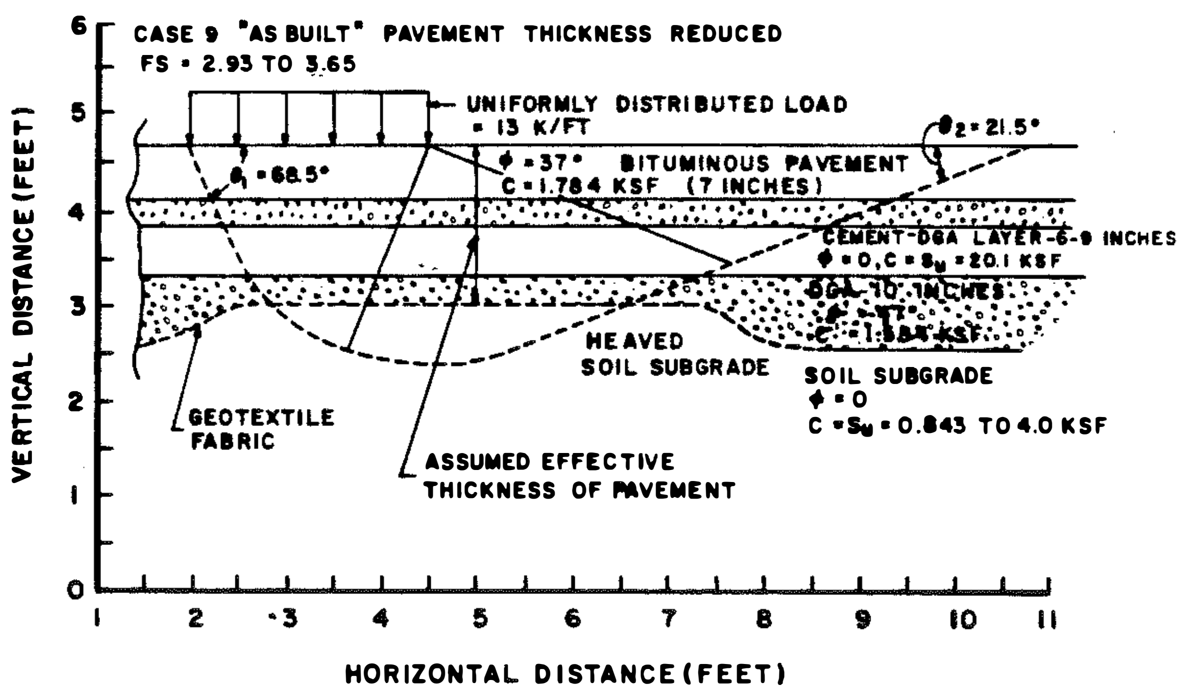

Figure 9. Assumed Failure Mode of the Remedial Pavement Assuming the Bottom 10-inch Layer of Dense-Graded Aggregate Is Effectively Reduced to 4 Inches.

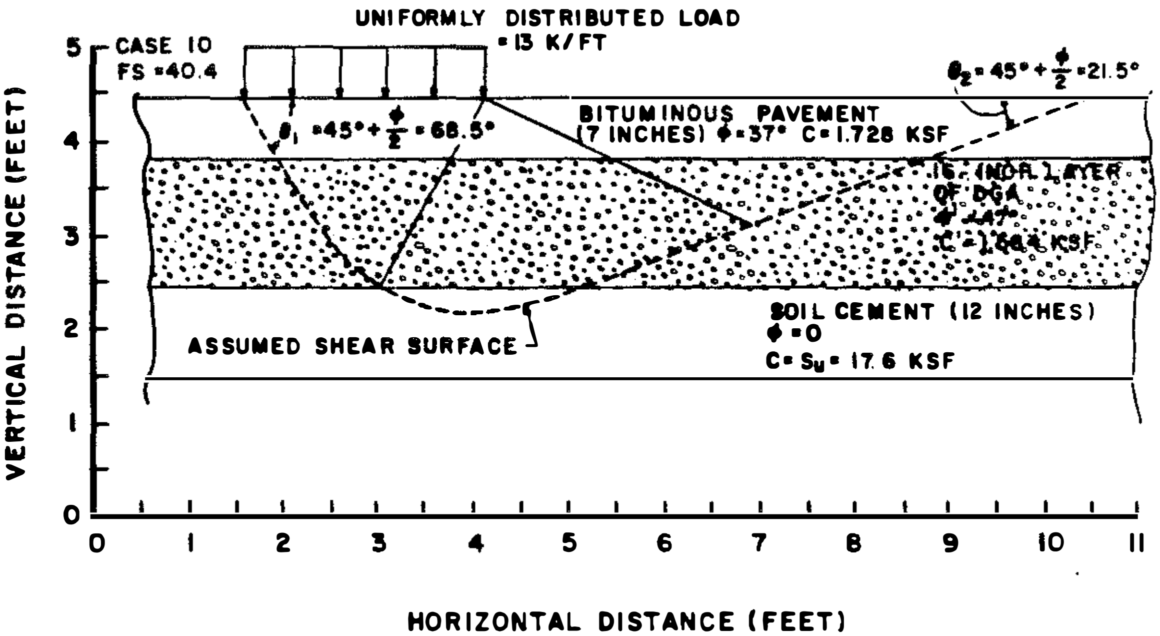

Figure 10. Assumed Failure Mode of 7 Inches of Bituminous Concrete and 16 Inches of Dense-Graded Aggregate on a 1-Foot Layer of Soil-Cement.

not available for the DGA. To obtain undrained shear strengths of the DGA, unconsolidated-undrained tests would have been required. Undrained shear strengths of the subgrade were available as shown in Table 4. Absence of pore pressures in the DGA material would occur only when no moisture was present in the DGA material. However, this is seldom the case. DGA is normally compacted at a water content of approximately five percent. Water may enter the DGA from surface runoff and subsurface seepage from springs and from underlying geologic formations. Hence, some pore pressures usually will be present in many situations.

The total pore pressure, u_{TDGA}, acting within the DGA layer, may be expressed as

$$u_{TDGA} = h_{el}/\gamma_w + u = h_{el}\ \gamma_w + u_e + u_{ss} \tag{1}$$

where h_{el} = elevation head

γ_w = unit weight of water,

u_e = excess pore pressure due to the application of traffic loadings and

u_{ss} = steady-state pore pressure.

If there is no flow and the pavement is relative flat (Figure 11), then

$$u_{TDGA} = u_{ss} + u_e \tag{2}$$

and

$$u_{TDGA} = u_s + u_e, \tag{3}$$

where u_s = the static pore pressure.

If there are no loads, then the excess pore pressure is zero. However, with the application of load, the excess pore pressures increase instantaneously since the pavement is usually loaded instantaneously. The magnitude of the excess pore pressure, u_e, is dependent on the permeability of the DGA, the degree of saturation, and the magnitude of the applied loads. If the DGA contains a small percentage of material passing the 200 sieve (less than or equal to five percent) and assuming the DGA is saturated, the magnitude of the excess pore pressure may be insignificant, provided water can flow freely at the sides of the DGA layer. However, if the DGA contains a substantial amount of material passing the No.-200 sieve (ten percent or more) and the material is saturated, then significant excess pore pressures may occur. When the fines passing the No.-200 sieve exceeds five percent, then significant internal movement of the fines may occur. Those fines generally move laterally under bituminous pavements and away from the point of application of the traffic loads. Movement of fines occurs as a result of dissipation of excess pore pressures; that is, water will flow laterally. Movement of fines laterally from the point of loading may cause a volume change in the DGA lying directly under the point of application of loads, and the pavement will tend to settle along the path of wheel tracks. Thus, rutting in the wheelpaths is the eventual result.

The permeability of DGA containing ten percent or more fines passing the No.-200 sieve is several orders of magnitude lower than the permeability of DGA containing five percent or less passing the No.-200 sieve. If there is a significant build-up of excess pore pressure, then the shear strength of the DGA, hence the stability of the pavement, is lowered significantly with application of loads. The shear strength, S_{DGA}, is dependent on the effective stress, or

$$S_{DGA} = c' + \sigma' \tan \phi' \quad (4)$$

where c' = effective stress cohesion,

σ' = effective stress = $\sigma - u$, (5)

ϕ = effective stress angle of internal friction,

σ = total stress, and

u = pore pressure in the DGA.

The effective stress is a function of the applied stress, Δq_s, and stress due to the weight of the material, q_i, above the point under consideration, or

$$\sigma = q_i + \Delta q_s \quad (6)$$

where $\sigma' = (q + \Delta q_s) - (u_s + u_e)$. (7)

The shear strength may be expressed as

$$S_{DGA} = c' + (q_i + \Delta q_s - u_s - u_e) \tan \phi'. \quad (8)$$

Stress increases at a given depth (Δq_s) due to the applied stresses at the ground surface must be determined from elastic theory (2, 11, 12). Hence, if the excess pore pressure is large, then the shear strength of the DGA is lowered. Consequently, the factor of safety is lowered. When the degree of saturation of the DGA is less than about 90 percent, then the excess pore pressure will be small. In that case, the shear strength will not be lowered significantly. Hence, as shown by Equations 1 through 8, continuous removal of water from the DGA layer is essential.

An expression similar to Equation 8 may be developed for the shear strength of the subgrade:

$$S_s = c_s' + (q_i + \Delta q_s - u_s - u_e) \tan \phi'_s . \quad (9)$$

When the stress, Δq_s, in the subgrade is large, then significant excess pore pressures may be generated. Usually, pavements are constructed on compacted fine-grained soils. Those materials generally are placed near optimum water content and maximum dry density. Water may eventually enter the subgrade from underlying geologic formations and seepage from surface runoff. The degree of saturation of the material when it is first compacted is usually about 85 percent. When stresses (due to traffic loadings) in the upper portion of the subgrade are sufficiently large, then the volume of the soil will decrease. With a decrease in volume, the degree of saturation increases. When the degree of saturation approaches 100 percent, and when the permeability is low, then excess pore pressures may be generated. The permeability of compacted fined-grained soils is very low (generally the coefficient of permeability may be 1×10^{-6} to 1×10^{-8} cm/sec). Dissipation of excess pore pressures occurs very slowly. With the build-up of excess pore pressures, the shear strength of the subgrade is lowered and the stability of the pavement decreases.

In the case of the I-65 failure, the upper 1-foot of the subgrade was compacted close to optimum water content and maximum dry density. However, the underlying soils were apparently wet and saturated. Moreover, water could enter this layer from surface runoff before the pavement was placed. Compacted fine-grained soils have a tendency to absorb water when the liquidity index is less than about 0.4 (7). Generally, compacted fine-grained soils have a liquidity index less than 0.4. Consequently, the upper zone of compacted subgrade apparently became saturated. As the degree of saturation increased, excess pore pressures developed under construction loadings. With repeated construction loadings, the soil "softens" and loses shear strength.

Case 11 considers the stability of the pavement "as designed" and the influence of pore pressures (acting within the DGA) on the factor of safety. The DGA layer was assumed to be saturated. Excess pore pressures, u_e, were estimated from Skempton's pore pressure equation (8),

$$u_e = \Delta\sigma_3 + A_f(\Delta\sigma_1 - \Delta\sigma_3) \tag{10}$$

where $\Delta\sigma_3$ = horizontal stress induced by the applied stress,

A_f = pore pressure parameter at failure, and

$\Delta\sigma_1$ = vertical stress induced by applied stress.

Assuming an elastic halfspace and using elastic charts given elsewhere (9), the vertical stresses induced by the applied load as a function of depth were estimated. The vertical stresses as a function of depth are shown in Figure 11. The induced vertical stress at the midpoint of the DGA (from Figure 11) is about 4.4 kips per square foot. The induced horizontal stress at the middle of the DGA layer was estimated to be 2.34 kips per square foot. Various values of A_f were assumed. Since the DGA is compacted, then this material is overconsolidated. Consequently, the A_f pore pressure parameter is probably below 0.6 (values of A_f may be obtained from triaxial tests). It is estimated that A_f probably ranges from -0.5 to, perhaps, 0.6. Using a value of -0.5, the estimated excess pressure is

$$u_e = 2.34 + (-0.5)(4.42 - 2.34) = 1.3 \text{ ksf},$$

or a head of water equal to about 21 feet. If A_f equals zero, then u_e

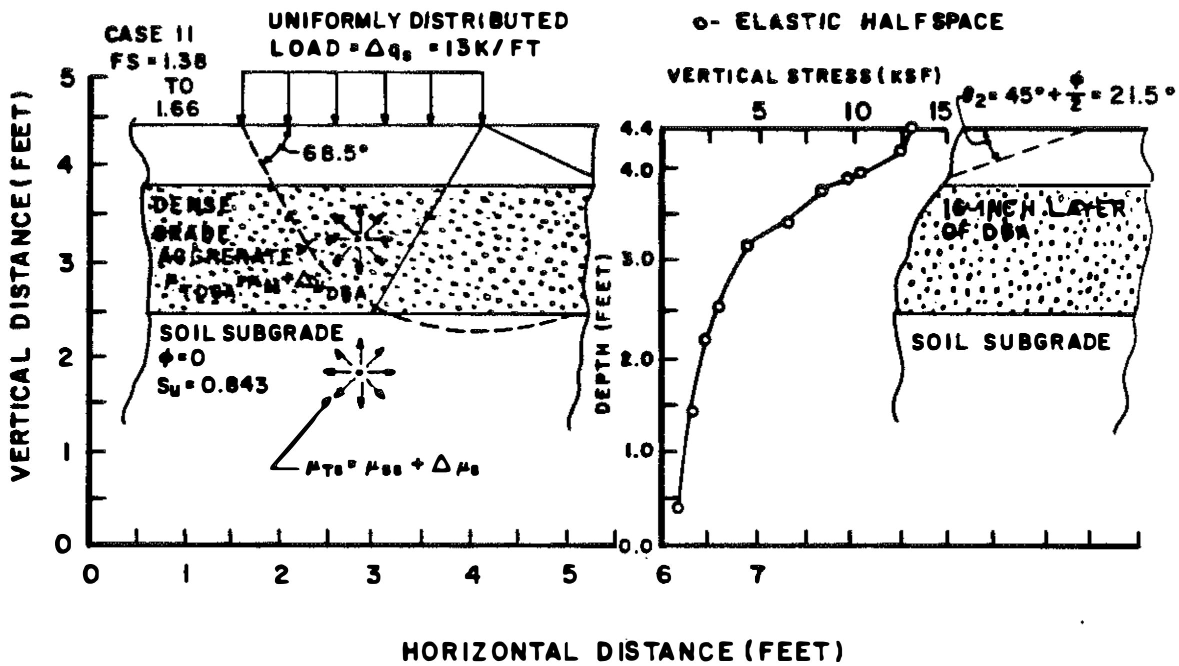

Figure 11. Pore Pressure Responses in Dense-Graded Aggregate and Subgrade; Vertical Stresses as a Function of Depth.

is equal to 2.34 kips per square foot, or a pressure head of about 37.5 feet. If A_f is equal to 0.6, u_e is equal to 3.59 kips per square foot , or a pressure head of about 58 feet. To enter these pore pressures into the HOPK-I stability program, the excess pore pressures were converted to pressure heads, which were entered into the computer program in the form of piezometric coordinates. In these analyses, it was assumed that the excess pore pressures develop only under the wheel loads. Factors of safety, corresponding to pressure heads of 0.0, 20.8, 37.5, and 60 feet, were 1.66, 1.56, 1.48, and 1.38, respectively. In these cases, the undrained shear strength of the soil was 0.843 kips per square foot. Hence, development of excess pore pressures in the DGA layer may lower the stability of the pavement. A similar analysis of the effects of pore pressures in the subgrade may be performed. Drained strength parameters of the soil subgrade were not available. The analyses above could be refined by performing triaxial tests on materials at a given site to obtain the necessary pore pressure parameters and using more sophisticated programs, such as the Chevron program (12), to obtain stress distributions. Moreover, in certain situations, it would be beneficial to monitor pore pressures in the DGA layer and subgrade to gain insight as to the magnitude of the excess pore pressures induced by traffic loadings.

ROAD RATER TEST RESULTS AND ANALYSIS

The Road Rater was used to obtain pavement deflections at three stages of construction in order to evaluate remedial measures:

1. Before stabilization of the upper 6-9 inches of DGA with portland cement,

2. After stabilization of the top 6-9 inches of DGA with portland

cement, and

3. After placement of 3 inches of DGA on the cement-treated DGA. The 3-inch layer of DGA was placed on the cement-treated DGA to eliminate or minimize potential reflective cracking of the bituminous concrete pavement. This condition is likely if the cement-treated DGA developed cracks.

The dynamic loading applied by the Road Rater was held constant at 600 pounds force dynamic and 1,670 pounds force static loading for all tests. Variations in measured deflections from one test series to another were attributed to the different construction stages. For example, if pavement deflections obtained after cement stabilization of the DGA were smaller than deflections measured before treatment, then the relative structural behavior of the pavement was assumed to have been improved. Deflection measurements were obtained at four locations relative to the point of application of the Road Rater load. Sensor No. 1 is located between the two "load feet" of the Road Rater (5.25 inches from the centroid of each "load foot"). Sensors No. 2, 3, and 4 are located at 1-foot intervals from Sensor No. 1. Deflection measurements obtained for the three construction stages are compared in Table 8.

Relatively small deflections obtained from the Road Rater are typically associated with high structural capacity of pavements. Larger deflections indicate the pavement may be structurally weak. Deflection values, as shown in Table 8, obtained after the upper 6-9 inches of the DGA layer was stabilized with cement were much smaller than deflections obtained before stabilization. Deflections decreased approximately 80 percent after treatment. Deflections obtained after the 3-inch layer of DGA was placed on the cement-treated DGA were nearly equal to or slightly greater than deflection values obtained from measurements on the cement-treated DGA. No significant changes occurred in the structural capacity of the pavement after placement of the 3-inch layer of DGA. Although the measurements indicate a very slight decrease in structural capacity after placing the 3-inch layer of DGA, the overall structural performance of the pavement was improved, since the 3-inch layer may prevent, or reduce, reflective cracking. Based on deflection measurements, the structural capacity of the pavement after treatment was significantly improved. The deflection values obtained after treatment are typically similar to values obtained for pavements that have historically performed well under similar traffic loadings.

The Chevron N-layer computer program (12) has been used to simulate Road Rater deflections and also has formed the basis for developing procedures for back-calculating the modulus of elasticity of pavement materials. Backcalculation procedures require simulation of Road Rater deflections for many conditions. Development of such a matrix was not considered necessary for this analysis. However, simulations were made for the three different construction stages (listed above) based on known layer strengths and assumed typical layer properties. Layer thicknesses, moduli of elasticity for each layer, and Poisson's ratio for each layer are required to simulate deflection values from the Road Rater. Layer parameters used to simulate theoretical deflections are summarized in Table 9.

Modulus of elasticity (in psi) of the subgrade may be estimated by multiplying the CBR value by 1500. The average CBR of the subgrade (Table 2) was 5.25. Therefore, the estimated modulus of elasticity for the subgrade is 7,875 psi. A Poisson's ratio of 0.45 was assumed for

TABLE 8. SUMMARY OF ROAD RATER DEFLECTION MEASUREMENTS AT DIFFERENT STAGES OF CONSTRUCTION, I 65, SOUTHBOUND LANES, STATION 120+00 TO 218+00

	DENSE-GRADED AGGREGATE											
	16 INCHES COMPACTED DGA				9 INCHES STABILIZED DGA 7 INCHES UNSTABILIZED DGA				3 INCHES UNSTABILIZED DGA 9 INCHES STABILIZED DGA 7 INCHES UNSTABILIZED DGA			
	SENSORS				SENSORS				SENSORS			
	NO.1	NO.2	NO.3	NO.4	NO.1	NO.2	NO.3	NO.4	NO.1	NO.2	NO.3	NO.4
STATION	(Inches x 10^{-5})				(Inches x 10^{-5})				(Inches x 10^{-5})			
139+50	246	234	211	196	37	26	15	10	55	24	15	10
145+00	252	240	224	196	40	27	14	10	46	25	18	16
150+00	240	234	217	195	41	33	22	16	48	23	18	15
155+00	258	240	224	197	62	38	27	18	68	44	25	17
160+00	243	230	217	197	28	19	9	7	32	22	14	9
163+50	255	246	226	199	74	45	24	18	59	38	25	15
171+50	242	238	221	199	33	25	18	10	32	23	12	8
Mean	248	237	220	197	45	30	18	13	49	28	18	13
Standard Deviation	7	5	5	2	17	9	6	5	13	9	5	4
Reduction of Deflections Relative to Measurements on 16 Inches of DGA												
Absolute Reduction In Means					203	207	202	184	199	209	202	184
Percent Reduction In Means					82	87	92	93	80	88	92	93
Theoretical Deflections at Mean CBR 5.25												
Std Deviation	148	90	59	42	101	80	58	42	86	73	56	42

TABLE 9. SUMMARY OF LAYER PARAMETERS USED IN THEORETICAL SIMULATION.

LAYER PARAMETER	16 INCHES OF DGA: DGA	16 INCHES OF DGA: SOIL SUBGRADE	9 INCHES OF CEMENT-TREATED DGA: CEMENT-TREATED DGA	9 INCHES OF CEMENT-TREATED DGA: DGA	9 INCHES OF CEMENT-TREATED DGA: SOIL SUBGRADE	3 INCHES OF DGA, 9 INCHES OF CEMENT-TREATED DGA, AND 7 INCHES OF DGA: DGA (3 In.)	CEMENT-TREATED DGA	DGA (7 In.)	SOIL SUBGRADE
Modulus of Elasticity (psi)	30,000	7,875	180,000	30,000	7,875	30,000	180,000	30,000	7,875
Poisson's Ratio	0.4	0.45	0.15	0.40	0.45	0.40	0.15	0.40	0.45

the subgrade.

Past experience indicates that the modulus of elasticity of DGA is typically in excess of 30,000 psi. A value of 30,000 psi was assumed in the analyses. The Poisson's ratio for the untreated DGA was assumed equal to 0.40. Samples of the cement-treated DGA were obtained from the construction site, molded, and cured for 7 and 14 days. Specimens were prepared following procedures described by ASTM D 1557(78), Method C. The unconfined compressive strengths and modulus of elasticity (static-chord method) of the specimens at the end of 7 days and 14 days were determined according to procedures of ASTM C 32(72) and ASTM C 469(65) (APPENDIX B), respectively. The static-chord modulus is the weakest modulus likely to be obtained since testing is conducted near a static loading condition and at high stress levels. The dynamic loading of the Road Rater is vibratory. Road Rater tests are conducted at much higher frequencies relative to unconfined compressive tests. The dynamic stress levels of the Road Rater are much smaller than the stress levels of the unconfined compressive tests. A Poisson's ratio of 0.15 was assumed for the cement-treated DGA.

Measured deflections were compared to calculated deflections using elastic layer theory and the Chevron N-layer computer program. These comparisons are shown in Table 8. Measured deflections of the 16-inch layer of DGA were much larger than deflections predicted from the theoretical simulation analyses. Deflections obtained after stabilization were considerably less than values obtained from the theoretical simulation analyses. Those results were not unexpected since the modulus of elasticity of the cement-treated DGA may be dependent on frequency of loading as noted previously. Additionally, stress dependent characteristics of granular base materials and soils also may have contributed to observed variations. Larger values of moduli of elasticity would be obtained from the Road Rater loading than moduli obtained from the unconfined compressive tests.

ESTIMATED FATIGUE LIFE OF PAVEMENT

The Kentucky 480-ksi thickness design curves (13, 14, 15, 16) were used to estimate fatigue life (in terms of EAL's) of the improved pavement and of the pavement as originally designed. Estimated fatigue life based on assumed subgrade CBR's of 3, 5, and 7 are compared in Table 10. Estimated EAL's of the original design were 9.0×10^5, 3.0×10^6, and 6.0×10^6 for CBR's of 3, 5, and 7, respectively, and were determined using the 480-ksi, 33-percent flexible pavement design curves. The same design curves were used to estimate the fatigue life of the improved, or modified, section and resulted in 3.0×10^6, 8.0×10^6, and 1.5×10^7 EAL's. EAL ratios (the ratio of EAL's for the modified design to EAL's for the original design) ranged from 2.50 to 3.33. Estimates of fatigue life of the modified section were determined on the basis of an increased total pavement section (23 to 26 inches with 33 percent asphaltic concrete) and do not reflect actual strengths of the cement-treated DGA or actual percentage of the total structure that is asphaltic concrete. Deflection analyses indicated that the cement-treated DGA restored the existing DGA to a structural condition equivalent to or exceeding the structural quality of a typical DGA base course (Tables 8 and 9). Additionally, compressive strengths and static-chord moduli of materials from the cement-treated section indicated much higher strengths than are typically obtained for unstabilized granular materials.

TABLE 10. COMPARISONS OF ESTIMATED LEVELS OF FATIQUE LIFE (EAL´S) FOR DIFFERENT VALUES OF CBR.

DESIGN CONDITION	LEVEL OF FATIQUE LIFE (EAL´S) CBR = 3	CBR = 5	CBR = 7
Initial Design 7 inches AC 16 inches DGA	9.0×10^5	3.0×10^6	6.0×10^6
Modified Design 7 inches AC 19 inches DGA	3.0×10^6	8.0×10^6	1.5×10^7
Ratio: Modified Design EAL´S / Initial Design EAL´S	3.33	2.67	2.50
Reduced Effective Thickness 7 inches AC 10 inches DGA	8.0×10^4	2.2×10^5	5.0×10^5
Ratio: Effective Thickness EAL´S / Initial Design EAL´S	0.88	0.73	0.83

The Chevron N-layer computer program was used to calculate critical strains in the pavement structure for comparison with limiting strains used in development of current flexible pavement design procedures. Critical strains were computed at the bottom of the asphaltic concrete layer and at top of the subgrade for two simulations. The first simulation involved the following:

E_{AC} = 480,000 psi, μ = 0.40, T = 7.0 inches

E_{DGA} = 30,000 psi, μ = 0.40, T = 16.0 inches

E_{SUB} = 7,875 psi, μ = 0.45, T = semi-infinite

Those values were selected on the basis of properties determined when comparing theoretically simulated deflection values to measured deflections. The subgrade value was further supported by CBR data previously presented. The 480,000-psi modulus was assumed as typical of asphaltic concrete pavements. Critical strains were calculated on the basis of 18,000-pound axleloads, dual tires, and 80-psi tire pressures. The resultant critical strains for this section were

E_{SUB} = 1.9165×10^{-4} in./in. vertical compressive strain at the top of the subgrade and

E_{AC} = 1.4281×10^{-4} in./in. tensile strain at the bottom of the asphaltic concrete.

Using limiting values presented in Table 1 and 2 and Figure 10 of Reference 13, a fatigue life of 4.0×10^6 to 5.0×10^6 EAL's may be projected. This estimate is in general agreement with values determined using the 33-percent asphaltic concrete thickness design curves for a CBR of 5.

A similar approach was used to estimate the expected fatigue life for the modified "as constructed" section. The following simulation was used:

$$E_{AC} = 480{,}000 \text{ psi}, \mu = 0.40, T_{AC} = 7.0 \text{ inches}$$

$$E_{DGA} = 30{,}000 \text{ psi}, \mu = 0.40, T_{DGA} = 3.0 \text{ inches}$$

$$E_{DGAS} = 180{,}000 \text{ psi}, \mu = 0.15, T_{DGAS} = 9.0 \text{ inches}$$

$$E_{DGA} = 30{,}000 \text{ psi}, \mu = 0.40, T_{DGA} = 7.0 \text{ inches}$$

$$E_{SUB} = 7{,}875 \text{ psi}, \mu = 0.45, T_{SUB} = \text{semi-infinite}$$

The 180,000-psi modulus for the cement-stabilized DGA layer was determined on the basis of static-chord modulus tests discussed previously. Critical strains for this condition follow:

$E_{SUB} = 1.8576 \times 10^{-4}$ in./in. vertical compressive strain at the top of the subgrade and

$E_{AC} = 1.0183 \times 10^{-4}$ in./in. tensile strain at the bottom of the asphaltic concrete.

Using the limiting values presented in Table 1 and 2 and Figure 10 of Reference 13, a fatigue life on the order of 2.0×10^7 EAL's may be anticipated for the "as constructed" section. This is somewhat greater than the projected failure estimates presented in Table 10 for a CBR 5 subgrade.

Each of these analyses do, however, apparently indicate that some significant benefit may be expected from the remedial actions.

In the bottom portion of Table 10, the EAL values are estimated for the original design assuming the 16-inch layer of DGA is reduced to 10 inches because of initial bearing capacity failures in the DGA base course. The ratio of EAL values for the reduced, or effective, thickness to the EAL values of the initial pavement design ranges from 0.73 to 0.83. Consequently, if no repairs had been made, the pavement would have developed problems prematurely.

CONCLUSIONS AND RECOMMENDATIONS

Based on the analyses presented herein, the following conclusions, recommendations, and opinions are presented:

1. Failure of the granular (dense-graded aggregate) base course during construction of the I-65 southbound lanes was a result of a bearing capacity failure in the clay subgrade. Moisture contents of the subgrade soils were greater than the optimum moisture contents. The undrained shear strengths of the subgrade were too low to support heavy construction equipment during initial construction stages. When the soils were loaded by heavy construction equipment, excess pore pressures developed rapidly because of the low permeabilities of the subgrade soils. When this occurred, effective stresses in the subgrade soils were reduced to, or nearly to, zero; the shear strengths of the subgrade soils were, consequently, reduced to essentially zero. Consequently, the subgrade soils failed. Based on the stability analyses, a factor of safety of about one was obtained for the case that considered 16 inches of DGA resting on the clay subgrade. Rutting and cracking of the DGA was observed at this stage of construction.

2. Remedial measures consisted of mixing about 7 to 9 percent portland cement with the top 6 to 9 inches of the DGA and placing a 3-inch layer of DGA on the top of the cement-treated DGA layer to prevent possible reflective cracking. These remedial actions were essential to assure proper performance of the pavement. Both stability analyses and Road Rater analyses indicated the premature failure of the pavement would occur if preventive measures were not implemented during construction. Analyses showed that the remedial actions should be very effective.

3. The dense-graded aggregate did not meet gradation specifications, although it may have met gradation specifications before placement. Samples of the DGA were obtained after placement and some degradation probably occurred during compaction. Generally, about 15 percent of the material passed the No.-200 sieve. Maximum and minimum specification limits are 12 and 5 percent, respectively. The DGA was generally compacted according to specifications.

4. The type of geotextile used at this location did not provide any significant pavement reinforcement since it was not a reinforcement-type material. However, the fabric apparently did prevent the intrusion of clay particles into the bottom portion of the DGA. Intrusion of clay particles would have significantly lowered the shear strength of the pavement and the DGA base layer. The fabric did not prevent failure of the DGA layer and soil subgrade under construction equipment. During the mixing of the cement with the upper 6 to 9 inches of the DGA, the fabric was reportedly cut at several locations. This condition could occur only when the subgrade had failed during early construction stages and moved upward relative to sections of the pavement that did not fail.

5. Pavement failures of the type that occurred at this site may be related to the low permeabilities and shear strengths of the fine-grained subgrade soils and saturated or near-saturated dense-graded aggregate. In fine-grained subgrade soils, excess pore pressures may develop instantly under heavy loadings; the low permeability tends to retard their dissipation. Large pore pressures reduce shear strengths of the DGA and subgrade; consequently, the stability of the pavement is lowered. Water moves from the subgrade into the DGA due to the excess

pore pressures. Hence, to prevent movement of soil particles into the DGA, the DGA must meet protective filter requirements. An exact check to determine if the DGA meet protective filter requirements could not be made since gradation data were not available for the subgrade soils. However, using a typical gradation curve of the soil in the project area, the DGA used at the site generally met filter requirements. To avoid internal movement of fines in the DGA, no more than 5 percent of the DGA should pass the No.-200 sieve. Since the DGA contained about 15 percent passing the No.-200 sieve, the fines will move internally or "pump" and tend to move laterally. The fines may move vertically through cracks and joints in the pavement.

6. Apparently, present design practices tacitly assume that a pavement "as designed" may be constructed. However, if the subgrade soils fail under construction loads during placement of the granular (DGA) base, then the effective thickness of the pavement "as designed" may be reduced in certain portions of the pavement, as visualized in Figure 3, due to heave of the subgrade. In other sections, the actual thickness may be greater than the design thickness. Hence, either the total thickness must be increased to compensate for the reduction or some form of subgrade stabilization must be implemented to prevent a bearing capacity failure of the subgrade. In certain cases, reduced construction loading might be used during construction of the first and second lifts of the granular base. Each case must be analyzed individually to establish safe loading limits for each stage of pavement construction.

7. One technique used on some pavement sections on the project consisted of mixing and stabilizing the subgrade soils with portland

cement. An analysis of pavements constructed on soil-cement stabilized subgrades yielded factors of safety in excess of 4.0. Hence, those pavement sections should perform adequately.

8. Bearing capacity analyses should be performed to assess the stabilities of pavements at different stages of construction. Bearing capacity analyses may be performed using Vesic's bearing capacity equations (2) for a two-layered system and/or the HOPK-I stability computer program (1). Undisturbed soil samples of the upper 3 feet of the subgrade would be required. Triaxial tests would be performed on those samples to define the undrained shear strengths of the subgrade. Alternatively, shear strengths of the subgrade could be established by performing triaxial tests on remolded, or compacted, soil specimens. Triaxial tests also would be performed on the bituminous concrete and dense-graded aggregate. Both unconsolidated-undrained triaxial tests and consolidated-drained or consolidated undrained triaxial tests with pore pressure measurements would be performed. In cases involving fine-grained soils such as clays or silts, the Dutch Cone Penetrometer could be used to rapidly obtain estimates of the undrained shear strengths of the upper 3 feet of the subgrade. Use of the Dutch Cone Penetrometer would require a relationship between Dutch Cone results and undrained shear strengths. Previous studies (10) have been performed in an effort to develop such a relationship. Additionally, deflection measurements obtained from the Road Rater might be used and correlated to the undrained shear strengths of subgrades. Deflection testing directly on subgrades has had some apparent success when correlated with CBR tests performed at in situ moisture contents. Where subgrades are constructed of granular materials, stability analyses would not be required.

9. The use of the HOPK-I stability computer program appears to be applicable to the evaluation of pavement designs for bearing capacity. Additional development and adaptation of this program is needed. An advantage of this approach to pavement design is that the mechanics of a critical mode of pavement stability (not normally considered) is modeled. This mode of failure is significant in early stages of construction of the pavement system. Total stress and effective stress analyses may be performed. Moreover, the shear strengths of the different materials in the pavement as well as the subgrade are used in the analyses. Short- and long-term stabilities may be studied. Generally, short-term stability is the most critical case for rapid loadings. Usually, the short-term stability is studied using undrained shear strengths in a total stress analysis. However, if pore pressures are known, or may be reasonably estimated, the short-term stability may be analyzed using an effective stress analysis, drained shear strengths, and estimated pore pressures. The method also may be used to design pavement overlays and pavement shoulders.

10. A granular base, such as DGA, must perform as a protective filter layer to reduce pore pressures below the surface pavement. To prevent intrusion of soils into the granular base, gradations of the granular filter and subgrade must meet filter requirements (11). Since the gradations of soils in the subgrade may vary widely from one region to another, gradation tests of subgrade soils should be performed to determine if filter requirements are met. Each case must be studied individually. In cases where the requirements are not met, consideration might be given to using a filter fabric.

11. Reduction, or elimination, of excess pore pressures in the DGA

layer and the subgrade soil is essential to prevent failure of pavements. Hence, good drainage is essential at all times. The build-up of excess pore pressures induced by heavy traffic loadings are particularly difficult to avoid. The use of side drains may be advantageous in draining the DGA material. As one means of aiding drainage in the granular base course, it is a good practice to slope the top surface of the subgrade toward each side of the pavement, that is, construct a subgrade "crown". However, side drains are not deep enough to drain the top portion of the subgrade. In subgrades constructed of fine-grained soils, such as clays or silts, some consideration might be given to measures that would aid in reducing excess pore pressures in the upper 2 or 3 feet of the subgrade. Perhaps as one means, the installation of wick drains in the upper 2 or 3 feet of the subgrade might be considered. Such a drainage system would aid in reducing excess pore pressures induced by heavy traffic loadings. This type of drainage also would aid in increasing the rate of consolidation of the subgrade soil. With an increase in the rate of consolidation, the shear strengths of the subgrade soils increase.

12. The design of shoulder pavements on the basis of bearing capacity may be particularly appropriate since the most severe shoulder loadings occur when a vehicle is parked.

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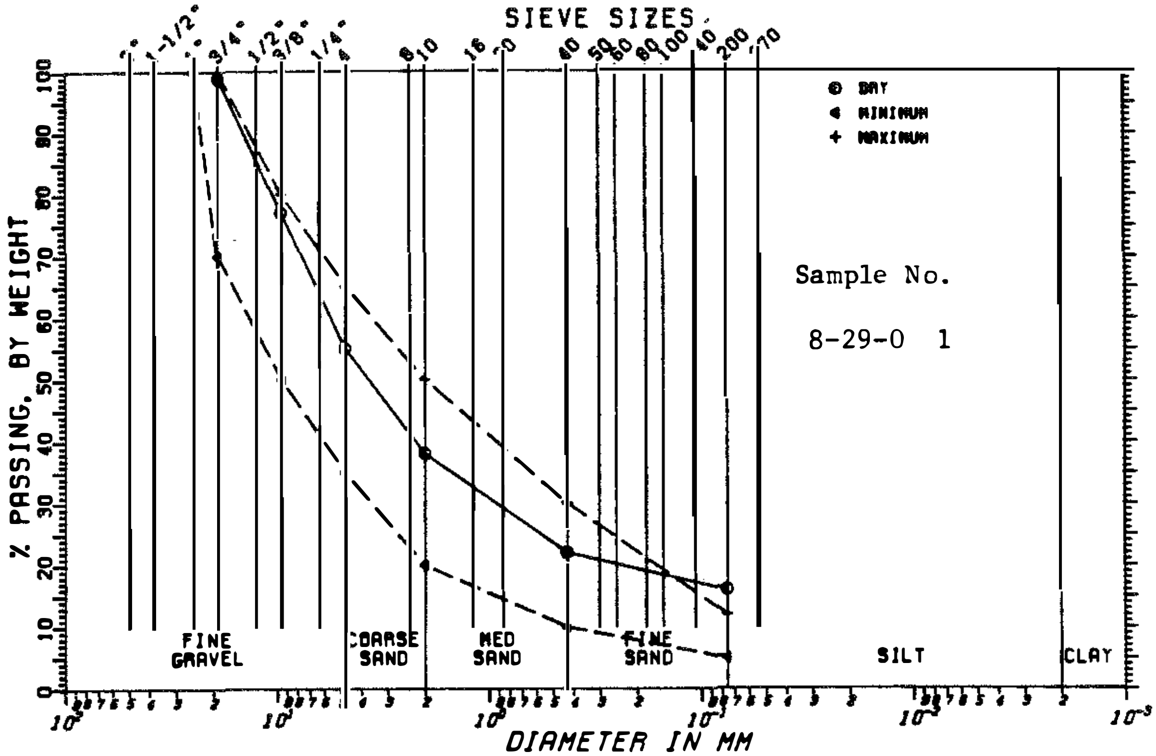
APPENDIX A

Gradation Curves -- Dense-Graded Aggregate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1-1/2"
1"
3/4"
1/2"
3/8"
1/4"
4
8
10
16
20
40
50
60
80
100
140
200
270
DRY
MINIMUM
MAXIMUM
Sample No.
8-29-0 1
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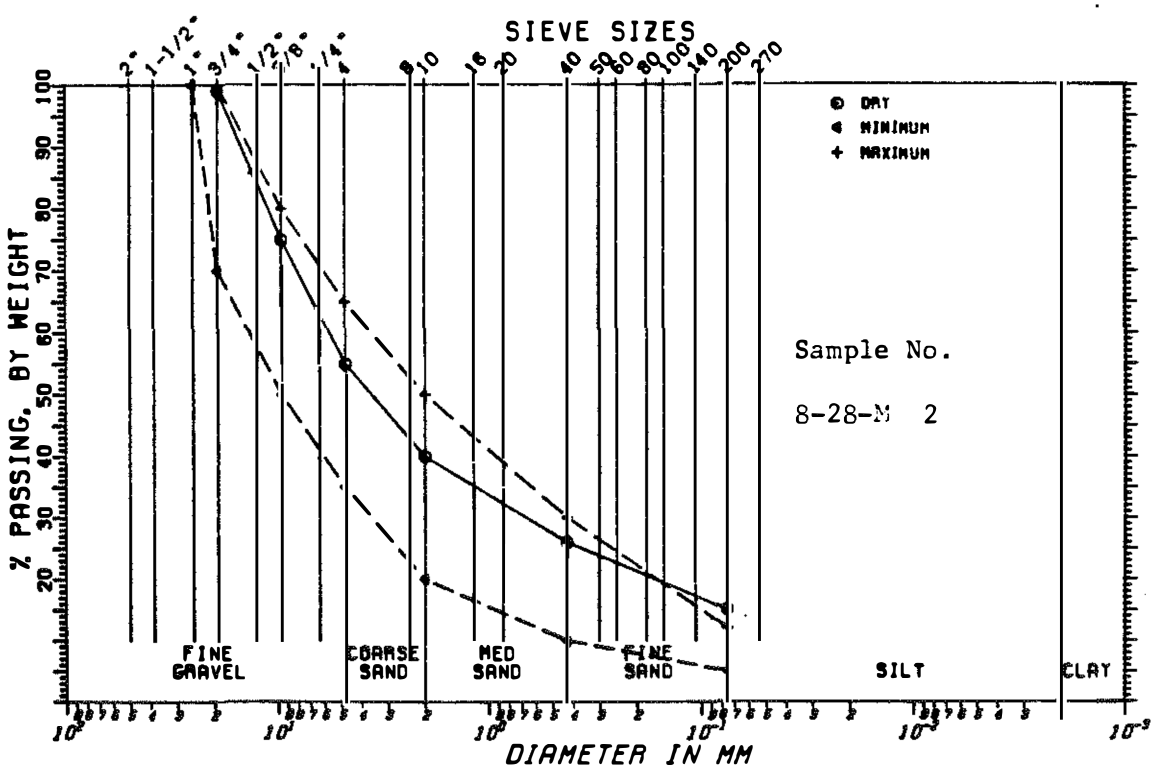
COARSE SAND
MED SAND
FINE SAND
SILT
CLAY
DIAMETER IN MM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3/4"
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3/8"
1/4"
4
8
10
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40
50
60
80
100
140
200
270
DRY
MINIMUM
MAXIMUM
Sample No.
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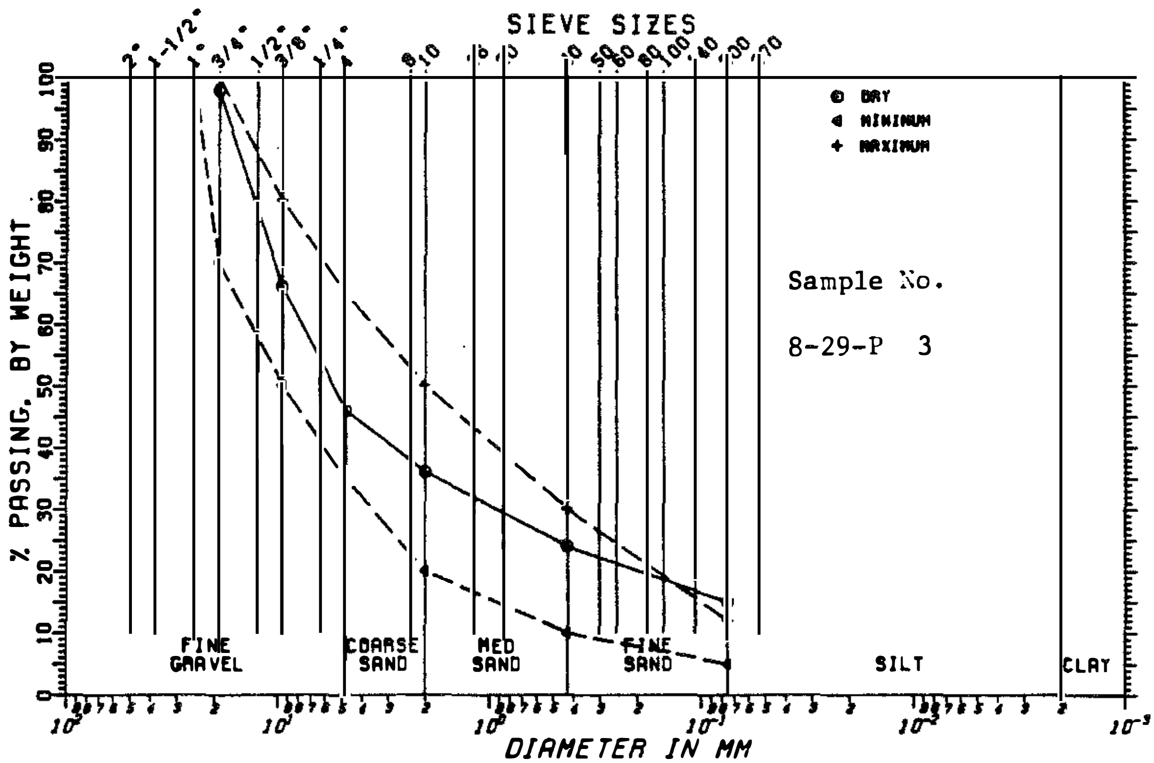
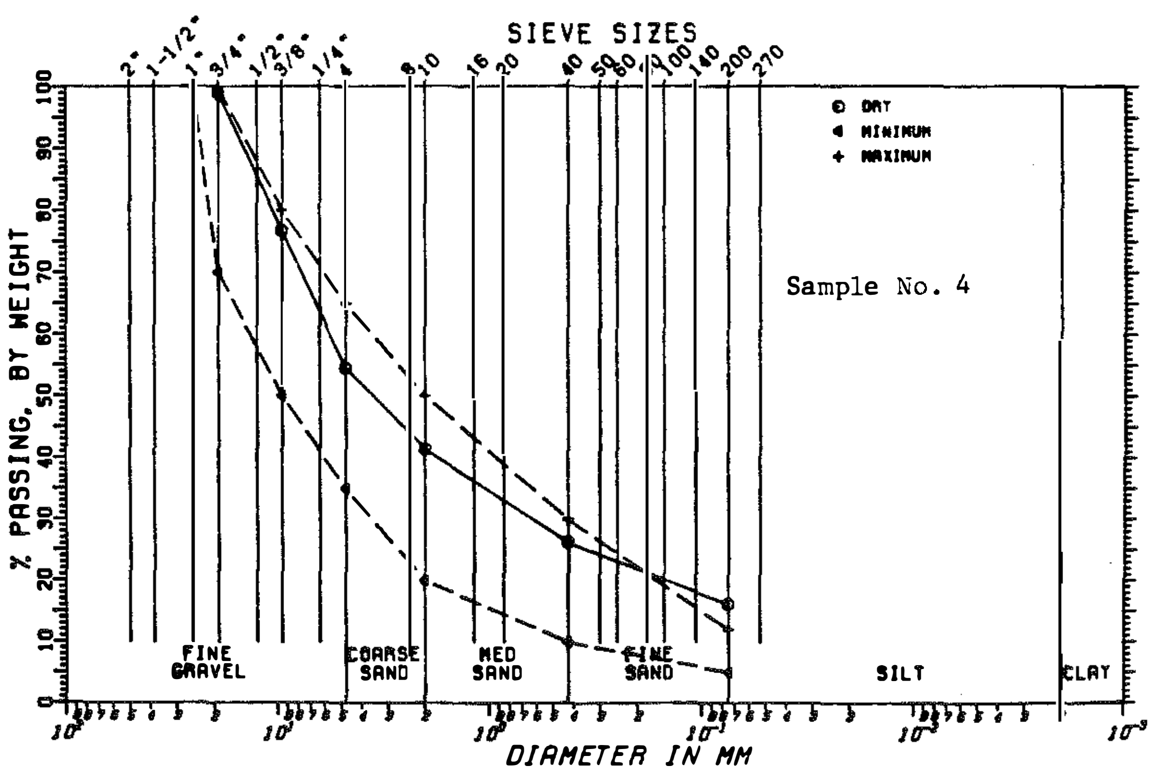
MED SAND
FINE SAND
SILT
CLAY
DIAMETER IN MM



SIEVE SIZES
2" 1-1/2" 1" 3/4" 1/2" 3/8" 1/4" 4 8 10 16 20 40 50 60 80 100 140 200 270
% PASSING, BY WEIGHT
DRY
MINIMUM
MAXIMUM
Sample No.
8-29-P 3
FINE GRAVEL
COARSE SAND
MED SAND
FINE SAND
SILT
CLAY
DIAMETER IN MM



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MINIMUM
MAXIMUM
Sample No. 4
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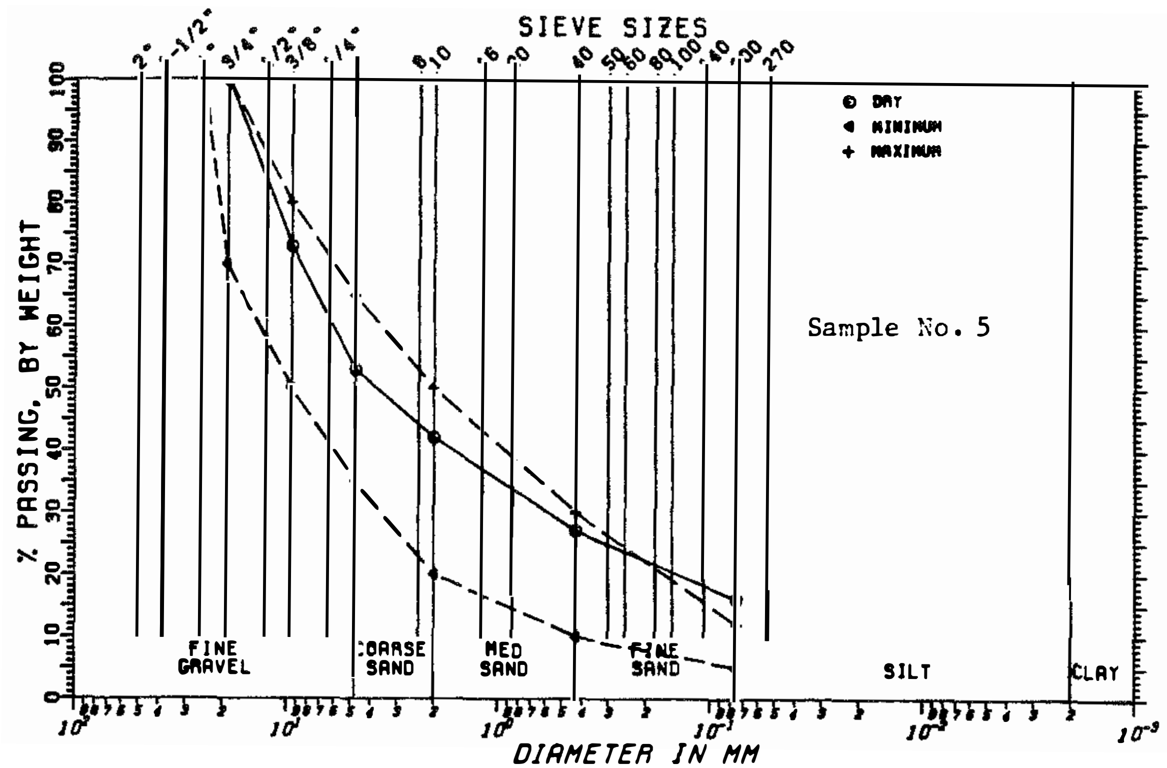
MED SAND
FINE SAND
SILT
CLAY
DIAMETER IN MM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% PASSING, BY WEIGHT
DIAMETER IN MM
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MAXIMUM
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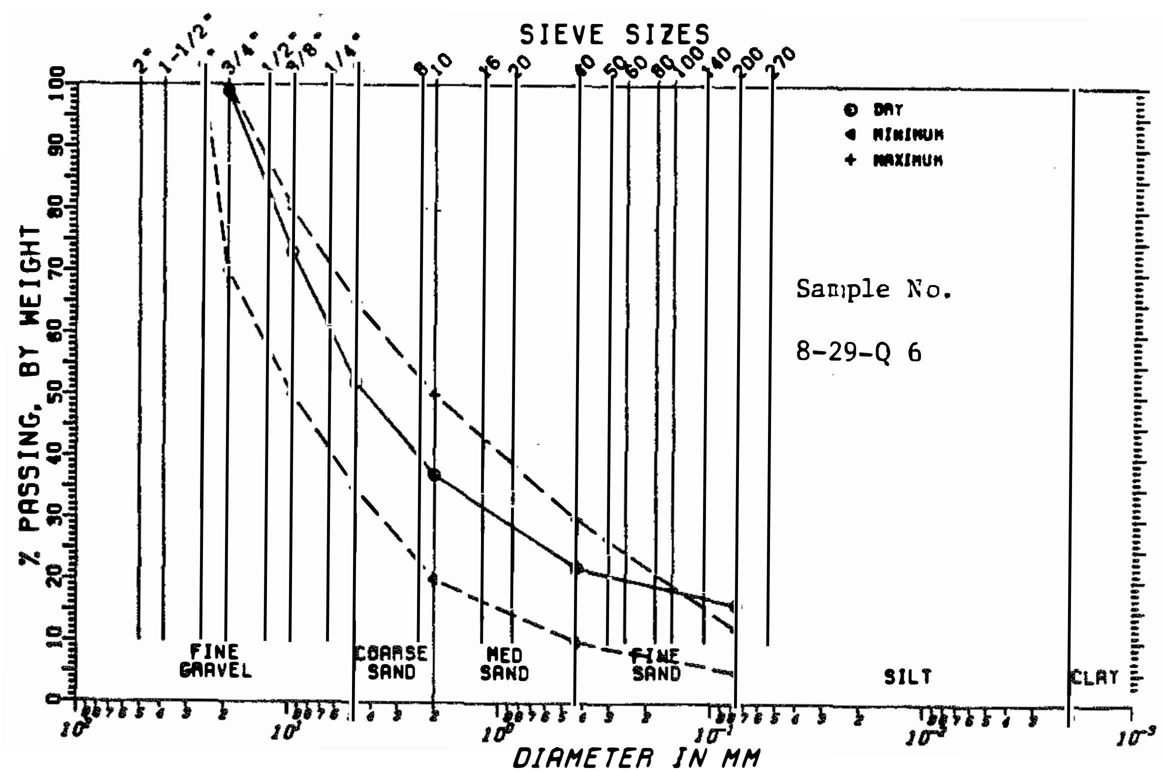
MED SAND
FINE SAND
SILT
CLAY
Sample No. 5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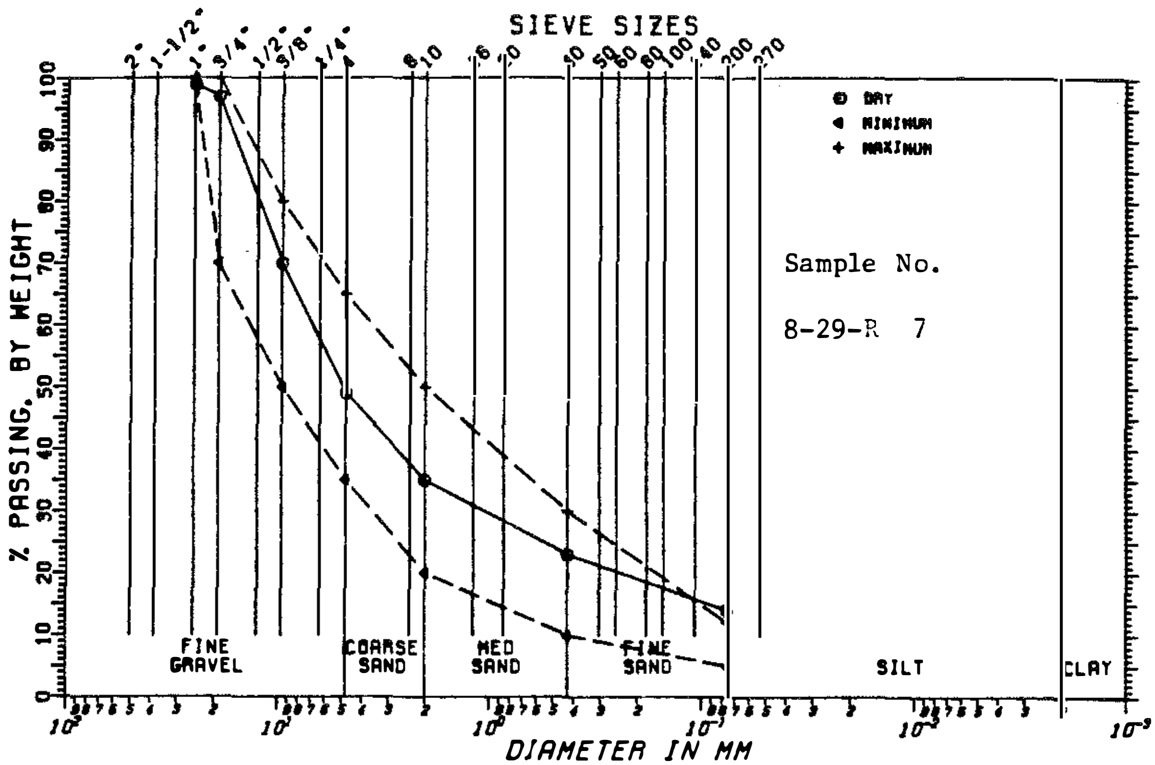
FINE SAND
SILT
CLAY
Sample No.
8-29-Q 6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MAXIMUM
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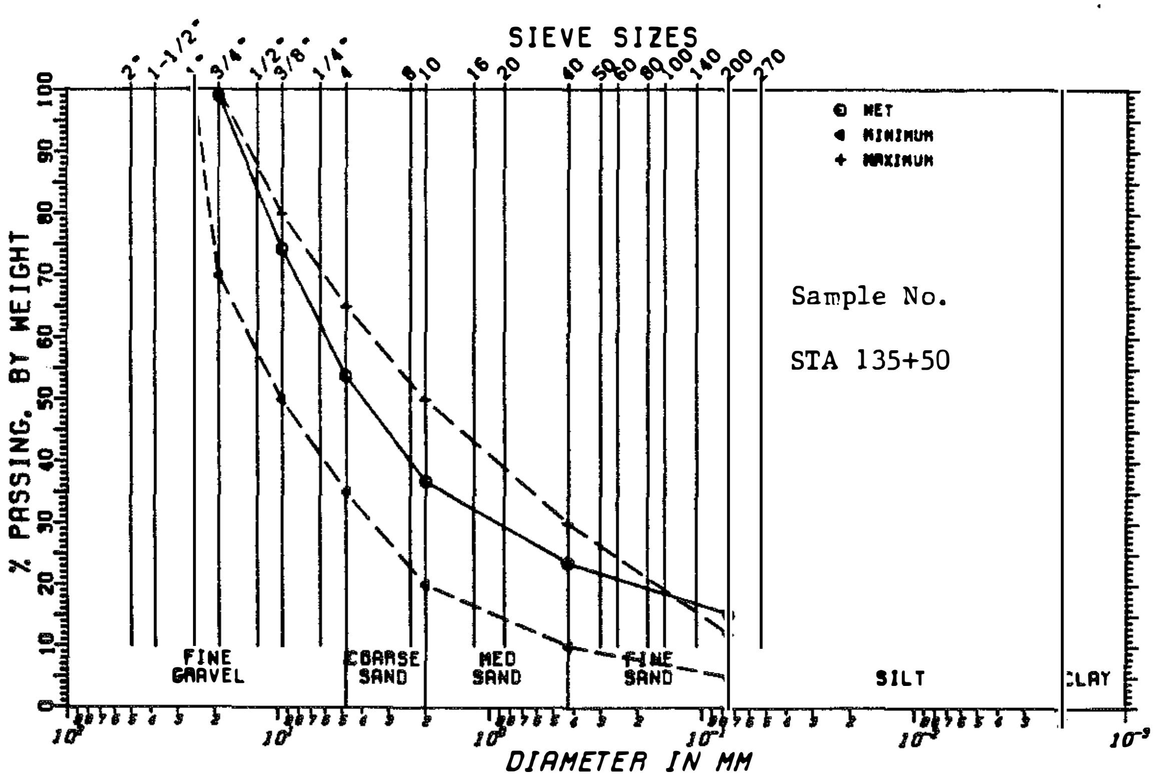
SILT
CLAY
DIAMETER IN MM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MINIMUM
MAXIMUM
Sample No.
STA 135+50
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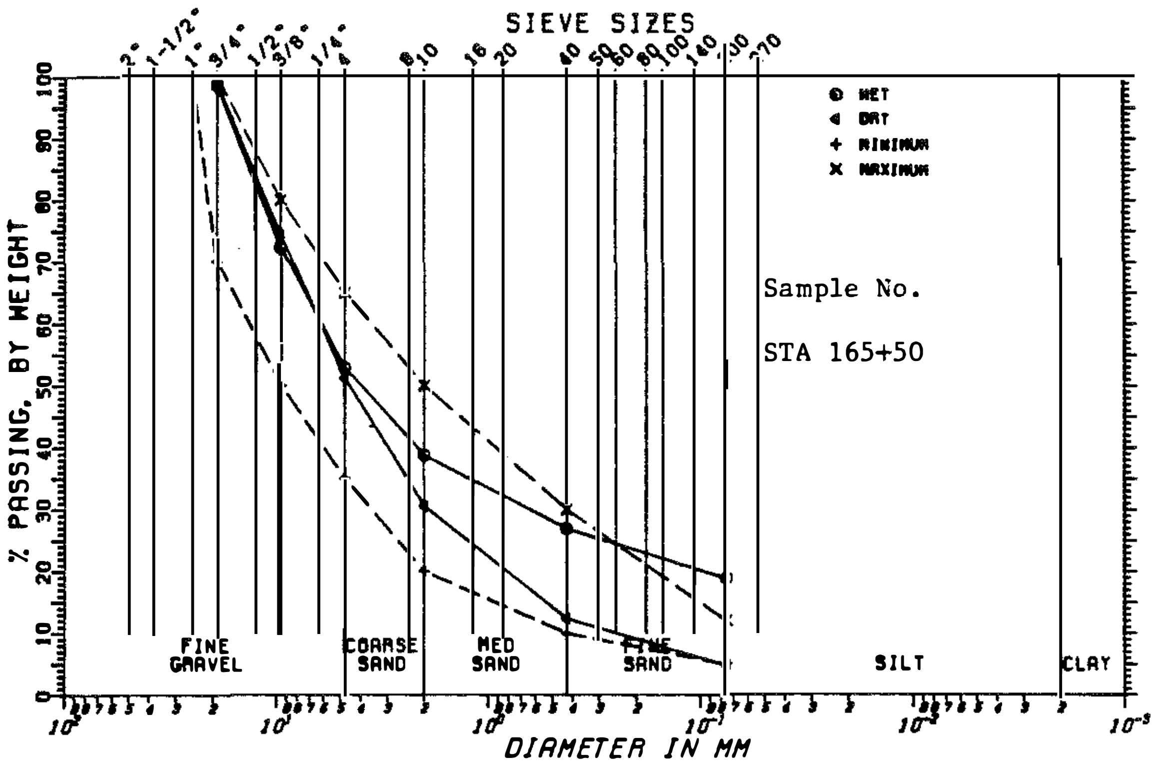
MED SAND
FINE SAND
SILT
CLAY
DIAMETER IN MM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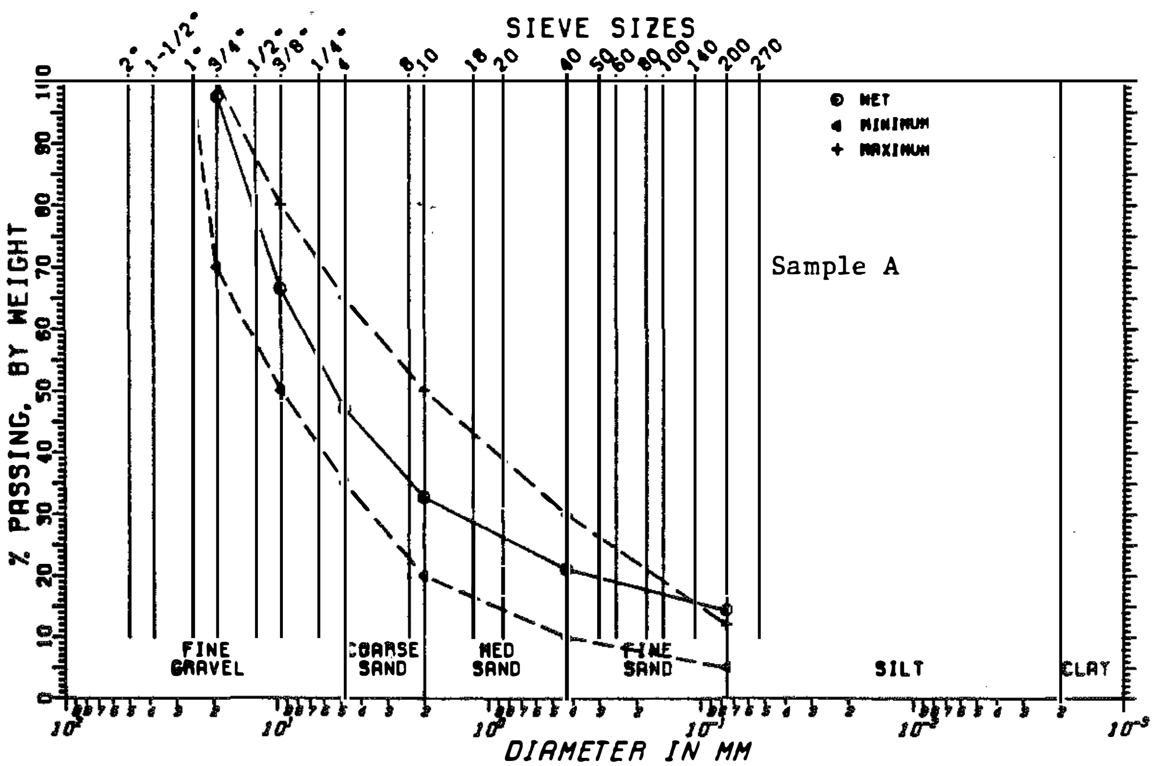
WET
DRY
MINIMUM
MAXIMUM
Sample No.
STA 165+50
% PASSING, BY WEIGHT
FINE GRAVEL
COARSE SAND
MED SAND
FINE SAND
SILT
CLAY
DIAMETER IN MM


SIEVE SIZES
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WET
MINIMUM
MAXIMUM
Sample A
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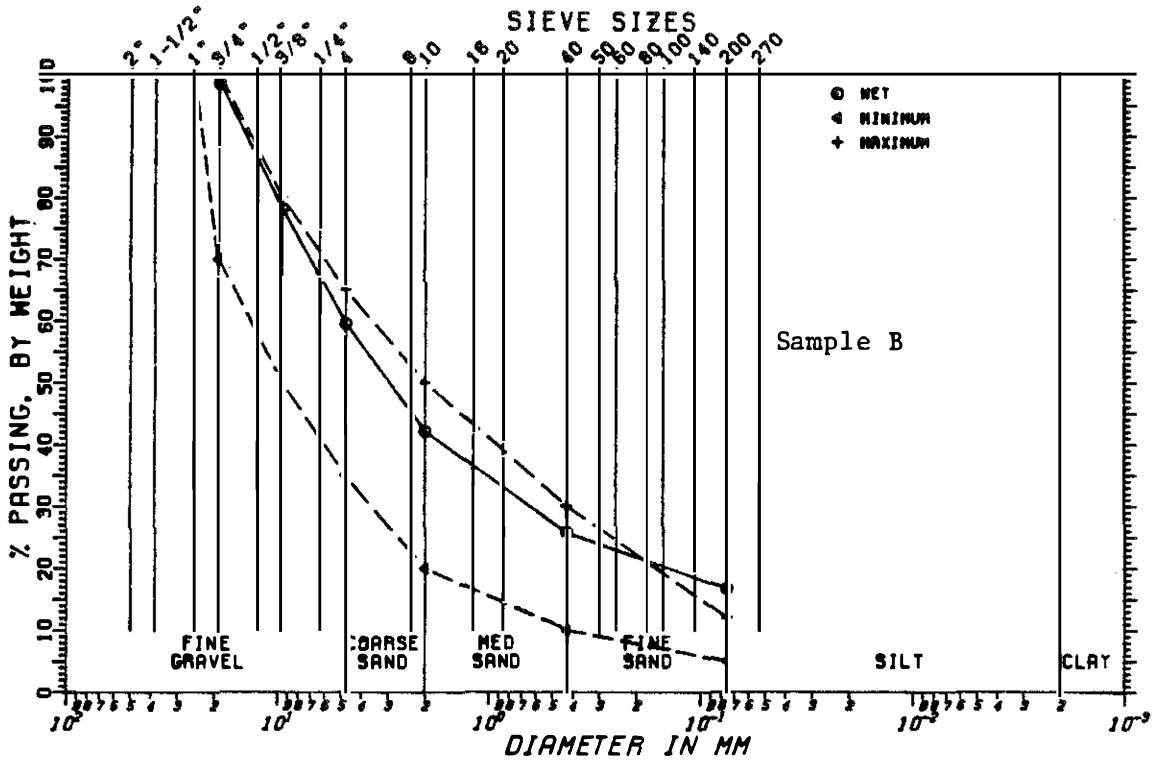
COARSE SAND
MED SAND
FINE SAND
SILT
CLAY
DIAMETER IN MM


SIEVE SIZES
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MINIMUM
MAXIMUM
% PASSING, BY WEIGHT
Sample B
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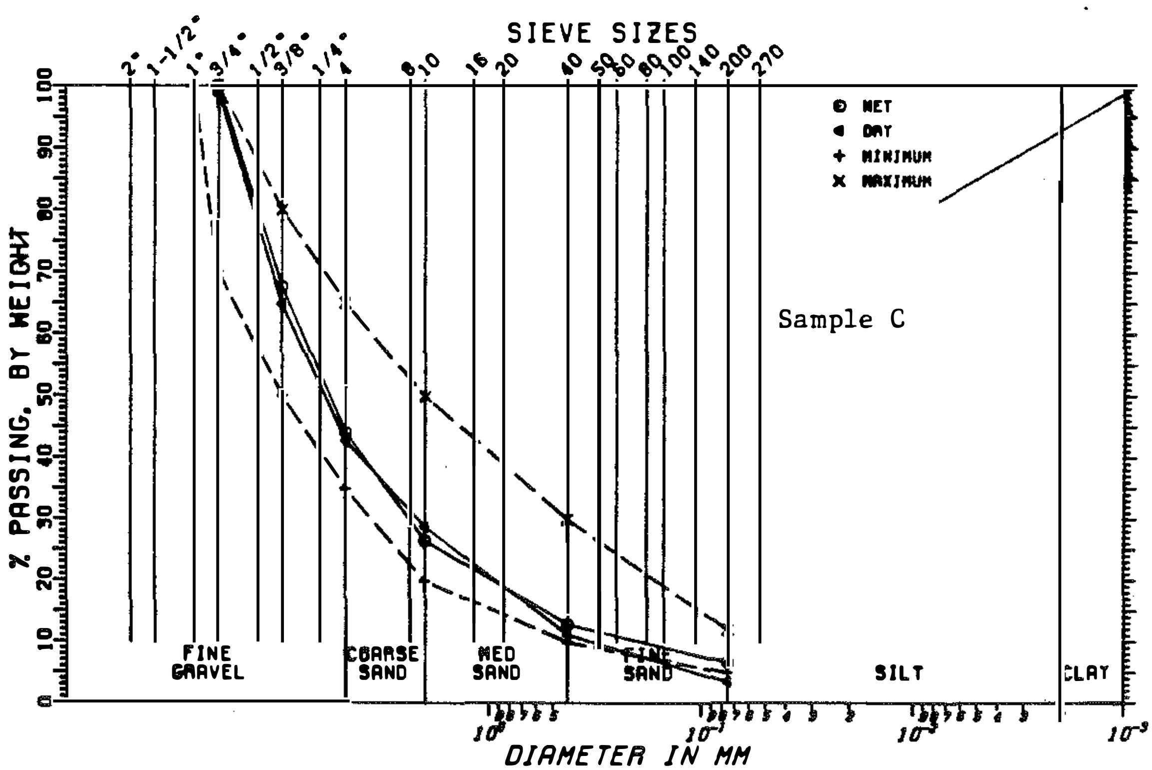
MED SAND
FINE SAND
SILT
CLAY
DIAMETER IN MM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2" 1-1/2" 1" 3/4" 1/2" 3/8" 1/4" 4 8 10 16 20 40 50 60 80 100 140 200 270
WET
DRY
MINIMUM
MAXIMUM
% PASSING, BY WEIGHT
Sample C
FINE GRAVEL
COARSE SAND
MED SAND
FINE SAND
SILT
CLAY
DIAMETER IN MM

APPENDIX B

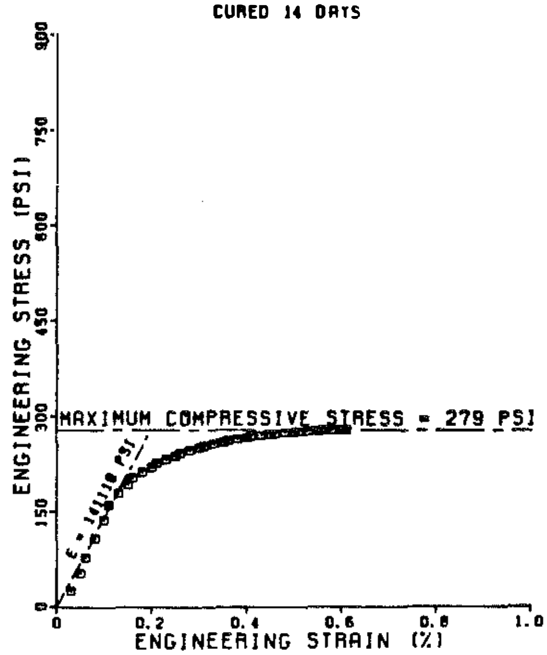
Stress-Strain Curves -- Cement-DGA Specimens

AXIAL STRESS-STRAIN PLOT

SAMPLE NUMBER : S-1

CEMENT TREATED BASE, I65 STATION 127+00

MIX CHARACTERISTICS
93 % DGA TREATED WITH 7 % CEMENT
MOISTURE CONTENT = 5.6 %
MADE 9-12-84
CURED 14 DAYS

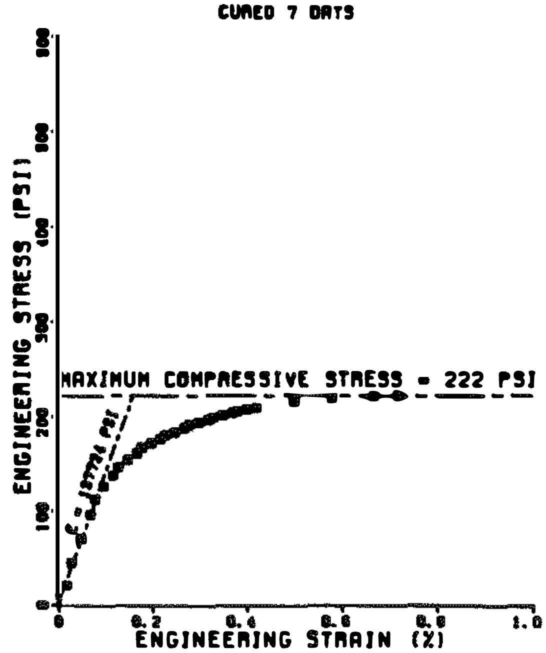


AXIAL STRESS-STRAIN PLOT

SAMPLE NUMBER : S-2

CEMENT TREATED BASE, I65 STATION 127+00

MIX CHARACTERISTICS
93 % DGA TREATED WITH 7 % CEMENT
MOISTURE CONTENT = 10.0 %
MADE 9-12-84
CURED 7 DAYS

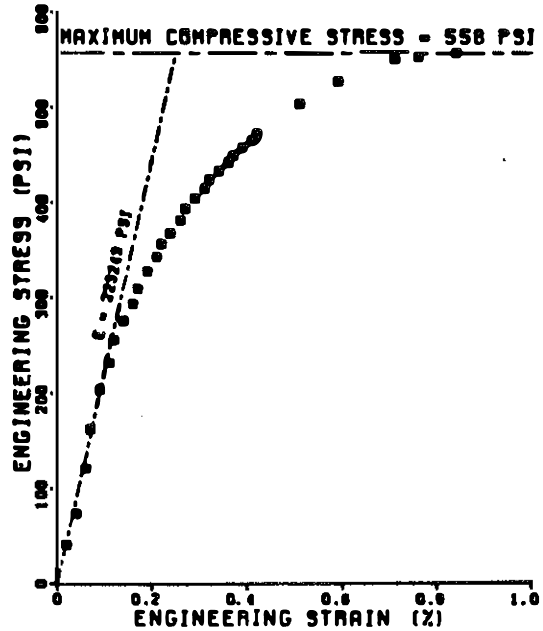


AXIAL STRESS-STRAIN PLOT

SAMPLE NUMBER : S-3

CEMENT TREATED BASE, I65 STATION 131+50

MIX CHARACTERISTICS
93 % DGA TREATED WITH 7 % CEMENT
MOISTURE CONTENT = 6.4 %
MADE 9-12-84
CURED 7 DAYS



AXIAL STRESS-STRAIN PLOT

SAMPLE NUMBER : S-4

CEMENT TREATED BASE, I65 STATION 131+50

MIX CHARACTERISTICS
93 % DGA TREATED WITH 7 % CEMENT
MOISTURE CONTENT = 3.8 %
MADE 9-12-84
CURED 14 DAYS