BEHAVIOR OF HIGH EARTH EMBANKMENT ON US-101

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Moisture and density requirements and zoning of three types of materials taken from roadway excavation were necessary for construction of a 383-ft highway embankment. Strength tests were performed on samples of the embankment obtained during construction to compare "as-built" strength with the strength values used in design. Compression within the embankment to date is on the order of 3 ft. Strength tests were performed on specimens measuring 12 in. in diameter by 27 in. high containing 3-in. maximum size particles. The maximum confining pressure was 400 psi. The triaxial test results obtained when subjecting a single test specimen to three increments of confining pressure compare favorably with the results when testing three separate specimens at different confining pressures. The materials used in construction proved to be stronger than assumed, thereby allowing a relaxation of the restriction on quality.

•IN THE north coastal region of California, US-101 traverses very rugged and unstable terrain. This terrain consists primarily of weathered sandstone and shale that are derived from the sedimentary rocks of the Upper Jurassic to Cretaceous age. The soils and rocks in this area constitute one of the largest masses of unstable material thus far encountered in highway construction in the State of California. The highway maintenance department and the traveling public have been aggravated for many years by numerous landslides and embankment slipouts that have closed the highway at times.

Planning engineers spent several years studying alternate alignments to determine the most feasible route for construction of a four-lane freeway with an all-paved section and design speeds of 60 to 70 mph.

To satisfy design standards of long radii curves and maximum grades of three percent, several embankments in excess of 250 ft in height were constructed. The embankment across Squaw Creek has a maximum height of 383 ft with variable side slopes ranging from 1½:1 to 2:1.

This paper deals primarily with the construction and performance of the Squaw Creek embankment and is a sequel to the paper by Hall and Smith (1), which emphasized the problems associated with the design of high embankments that are built with normal roadway excavation. This paper describes strength parameters obtained during construction. Also included is a discussion of behavior of the embankment, pore pressures, settlement, and modification of design. A procedure for determination of strength parameters for more than one lateral pressure applied to a single test specimen is presented.

CONSTRUCTION

Construction of the project began in June 1966. A major portion of a $6,000,000 contract consisted of placing approximately 2.5 million cu yd of roadway excavation in

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the Squaw Creek embankment. Weather conditions in this construction area compel winter shutdowns of approximately six months each year. Approximately one-third of the embankment material was placed during the 1966 construction season. The remainder of the embankment, except for the structural section of the roadway, was completed during the 1967 construction season. The road was opened to traffic in late 1968. Since that time the road has functioned satisfactorily with no signs of distress appearing in the embankment.

Foundation for the embankment consisted of fairly good weathered interbedded sandstone and shale. Some surface weathering had occurred to the extent that removal was necessary. Most of the removal was made in the bottom of the rather steep-sided creek or canyon. Depths of removal in no case exceeded 20 ft. Material was stripped to expose somewhat better quality weathered sandstone and shales. Several seepage areas were exposed in the bottom of the creek as well as on the flanks of the canyon. A 3-ft layer of permeable material, 20 to 50 ft wide, was placed in the bottom of the creek. Projections of permeable material extended from this system to many seepage areas that were exposed or existed in the foundation area. A perforated metal pipe was placed in the permeable material in the bottom of the creek. This subsurface drainage system has produced large quantities of water throughout the history of the project and continues to flow at a fairly high rate, particularly in the winter. Total flows have ranged from a maximum of approximately 500,000 gal/day to a minimum in the late summer of 63,000 gal/day. Observations indicate that this system has effectively prevented the development of pore pressure in the lower part of the embankment or in the foundation materials. Some investigation was done during construction to determine if groundwater or pore pressures existed. Generally, this information appeared favorable. There were some indications of local pockets of perched water, but these were believed to be of minor consequence.

At the planning and design stages it was decided to transfer the surface water normally carried by the creek to a culvert to be installed relatively high in the embankment and on one of the slopes of the canyon. The culvert was to extend from the top of the area designated "fill for drainage" through the fill and would discharge on the natural ground rather than the side of the embankment. To accomplish this task, additional roadway excavation was deposited in the area designated as "fill for drainage." Thus, the surface water and water in the creek were handled with a culvert of relatively light construction in contrast with an exceedingly heavy culvert that would have been required had it been placed in the bottom of the embankment.

Settlement platforms were installed in the embankment near original ground and at 50-ft vertical increments during the two construction seasons. Units were also installed on the downhill slope of the embankment during the 1967 construction season. A typical section showing vertical movement platforms and other construction control devices is shown in Figure 1. Settlement data to date indicate the embankment has compressed or consolidated approximately 3 ft, and the foundation soils have settled approximately 1 to 2 ft. Most of this settlement occurred in a period of about 1 yr during construction (1966-1967).

The indicating units installed on the downhill slope of the embankment show horizontal movements on the order of ½ to 1 ft and most of this movement occurred during the 1967 construction season.

Three general zones of material were used in the embankment. Figure 1 shows a cross section through the more massive portion of the embankment. Zone C was composed of the sounder and larger rocky material that was encountered in excavation of designated cuts and was relatively free of clay. No special compaction of this material was specified or required. The material, being relatively free of fines, compacted readily with heavy-duty hauling, spreading, and compacting equipment. Most of the material contained such a high percentage of plus ¾-in. material that standard compaction tests were impractical. Zone B consisted of selected rocky, granular material containing no material normally classified as overburden and containing no appreciable amounts of shale, clay, soil, or vegetable matter. The gradings of the material tested for design are shown in Figure 2.

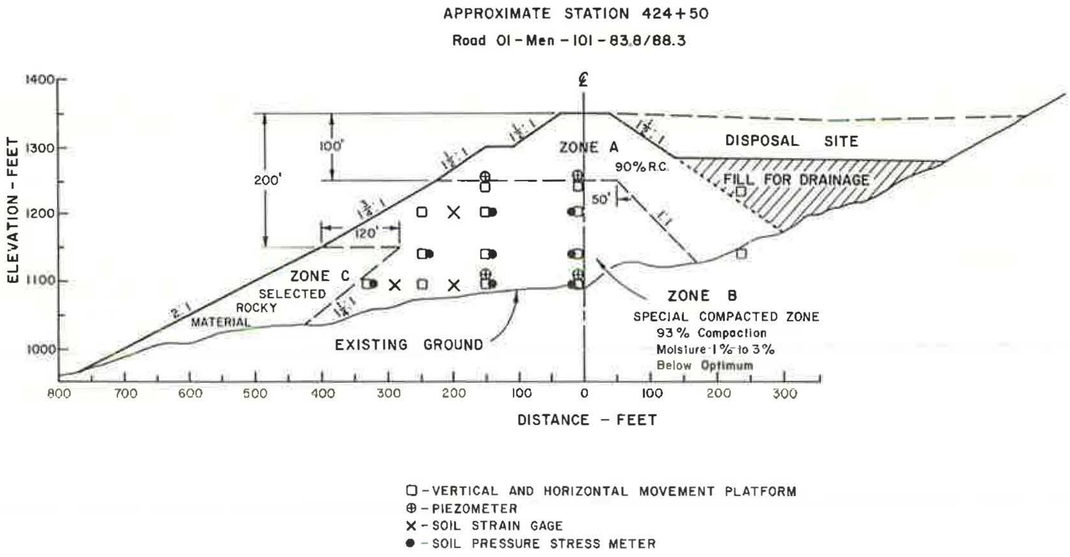


Figure 1. Typical section at Squaw Creek.

Specifications required that the Zone B material be compacted to 93 percent relative compaction. Test Methods Calif. Nos. 216 and 231 were used. The maximum density results from these test methods are comparable to results from AASHO T-188. The Corps of Engineers, who did the testing for design, used the AASHO test for determining maximum density. Specified moisture was 1 to 3 percent below optimum. Density requirements were met with reasonable compactive efforts. Some special efforts were

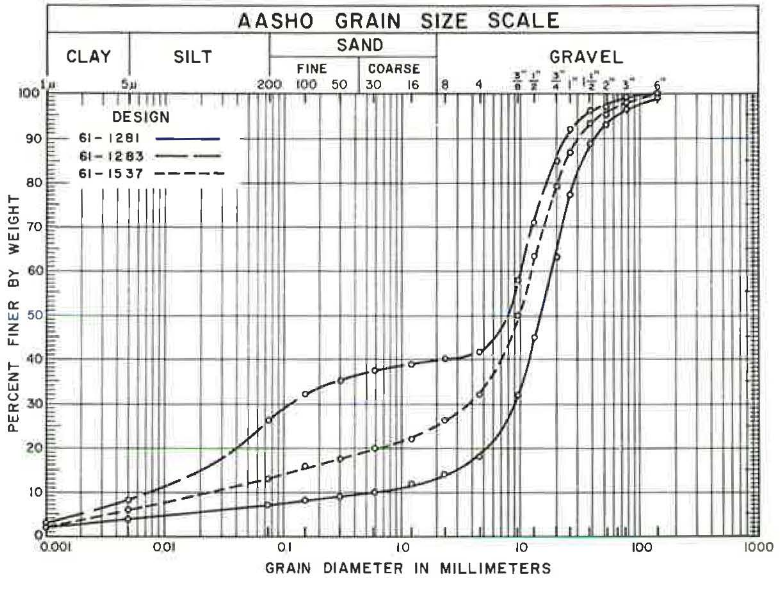


Figure 2. Mechanical analysis of material used in design.

required to lower the moisture content to the required level. Compaction and moisture requirements for Zone A material were in conformance with standard specifications for construction of California highways. The only moisture requirement contained in the California Standard Specifications for the construction of embankments states "the moisture content of embankment material shall be such that the specified relative compaction will be obtained." The normal compaction requirement is 90 percent relative compaction.

STRENGTH TESTS

The quantity of Zone C material was limited during the early months of construction. Visual inspection of material being placed in Zone B indicated that the material contained more clay than was anticipated during the design stage (Figs. 2, 3, and 4). Index properties of design samples are given in Table 1. The strength values used for design of the three construction zones are given in Table 2. Late in 1966 several large samples, approximately 2 tons each, were taken from the material being placed in the embankment. Gradings of these samples are shown in Figure 3. Based on visual observations, these samples were believed to be representative of the range in quality of material being placed. Test results from five of these samples are given in the left side of Table 3. This table also gives the results of tests performed on samples secured during the 1967 construction season. Grading curves of these samples are shown in Figure 4. A comparison of the strength test results (Table 3) and those used in design (Table 2) shows that the quality of the material being used in construction of both Zones B and C was slightly better than the values based on test results used for design. Shear strength of the Zone A material was consistently better than was anticipated. Consequently, restrictions on quality for materials in Zones B and C were slightly relaxed. It was possible to use material in these zones that was finer than was anticipated during design. Generally, the 1967 strength data show better quality material than was obtained in 1966. This is probably explained by the fact that, as the project neared completion, most of the material was being excavated from the lower portions of the cuts, and the quality of the material generally improved.

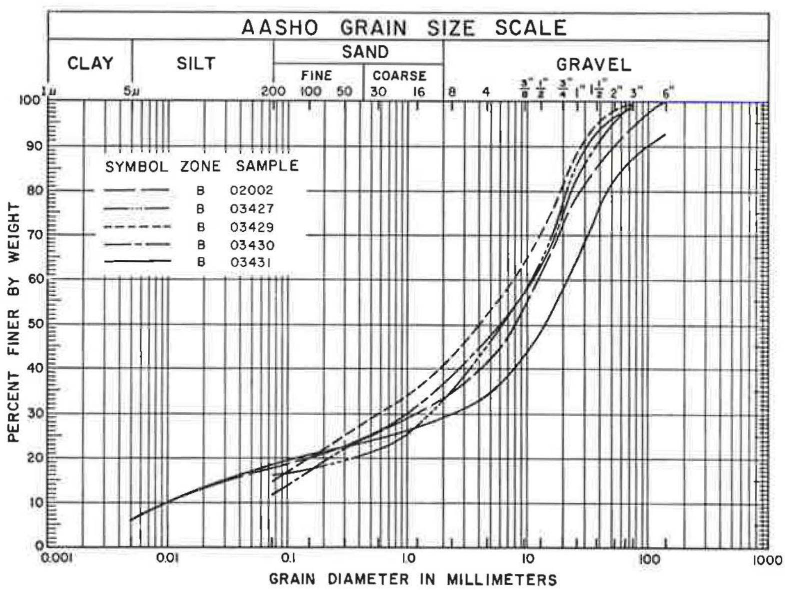


Figure 3. Mechanical analysis of material placed in embankment, 1966.

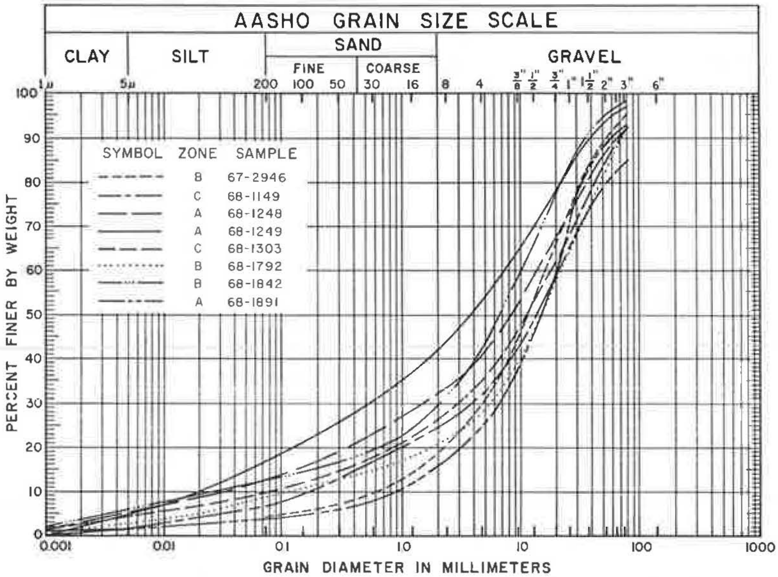


Figure 4. Mechanical analysis of material placed in embankment, 1967.

As a result of the favorable strength values obtained during construction and the satisfactory behavior of the embankment, it was agreed that unexpected slide material could be placed on the area designated "fill for drainage." This has been accomplished during the last 2 years subsequent to completion of the project. This area also served as a disposal site for excess material from additional nearby construction (Fig. 1).

TABLE 1

INDEX PROPERTIES OF PROPOSED EMBANKMENT SOILS

Description	Sample[a]		
	No. 61-1281	No. 61-1283	No. 61-1537
Identifying properties	Dark gray shale; very crumbly; breaks down readily on handling	Brown, highly weathered shale and sandstone; breaks down very readily	Gray sandstone and shale; appears rather durable, but also degrades; the best of the three samples
Liquid limit	45	28	28
Plastic limit	26	24	23
Plasticity index	19	4	5
Sand equivalent	22	14	20
R-value	33	40	81
Compaction (Mod. AASHO)			
Maximum density (pcf)	129.3	126.3	132.9
Optimum moisture (percent)	9.8	8.7	8.1
Strength data			
Effective stress			
Cohesion (psi)	5	11	20
Angle of internal friction (deg)	36	34	34
Total stress			
Cohesion (psi)	20	15	20
Angle of internal friction (deg)	28	33	33

[a]All samples compacted to 93 percent relative compaction at 2 percent less than optimum moisture.

SPECIAL TEST PROCEDURE

A significant requirement of any testing program is that the laboratory test specimen duplicate, as nearly as possible, the material and conditions in the field. Most conventional triaxial testing equipment is suitable only for testing specimens 4 in. or less in diameter that contain little or no plus ¾-in. fragments and exerting lateral pressures on the order of 80 to 100 psi. The California Division of Highways is now equipped to test 12-in. diameter by 27-in. high specimens with maximum lateral pressure of 550 psi. The test specimens for this program were of such size and contained 3-in. maximum size particles.

TABLE 2

STRENGTH VALUES USED FOR DESIGN OF CONSTRUCTION ZONES

Zone	Cohesion (psi)	Angle of Friction (deg)	Percent Compaction
A	3.5	15	90
B	3.5	25	93
C	0[a]	35[a]	95[a]

[a]Estimated values.

As stated previously, the selected rocky material was readily compacted to the required 93 percent relative compaction. Therefore, the triaxial test specimens were also compacted to 93 percent relative compaction and at the field moisture content that was generally in the range of 1 to 3 percent less than optimum.

A modification from the usual triaxial testing procedure was applied to some specimens whereby a single specimen was confined with three successively increasing lateral pressures. When the stress-strain curve showed that the sample was beginning to fail or had reached a predetermined strain, the lateral pressure was increased to the next increment and the procedure repeated. Tests were also performed by the usual procedure; that is, a separate specimen was used for each lateral pressure. The results of this testing program are given in Table 3. There exists a narrow range in variation between the usual procedure and the one-specimen procedure; in fact, one group gives nearly duplicate results. Stress-strain curves and Mohr envelopes are shown in Figures 5 through 15. Results from the two types of tests can be compared with the information given in Table 3. In general, for the material tested on this project, either method produced the same results. This duplication of test results is not necessarily true for all materials. Tests on other materials are being conducted. This technique results in extensive saving in time, money, and materials. Approximately 2 man-days of labor are required per specimen regardless of test procedure used, thereby appreciably increasing the testing capacity of the laboratory.

TABLE 3

INDEX PROPERTIES OF SOIL SAMPLED FROM EMBANKMENT

Description	Sample Identification (T-101 Number)													
	Sampled 1966						Sampled 1967							
	03426	03427	03429	03430	03431	02002	03432	03433	Cut A 02504	Cut B 02504	02505	05755	05754	05809
Test number	SC-1	B	C	E	F	5	67-2946	68-1149	68-1248	68-1249	68-1303	68-1792	68-1842	68-1891
Construction zone	B	B	B	B	B	B	B	C	A	A	C	B	E	A
Liquid limit	—	37	25	—	24	25	27	—	22	22	19	23	31	24
Plastic limit	—	23	18	—	16	18	18	—	20	16	18	19	19	17
Plasticity index	—	14	7	NP	8	7	9	NP	2	6	1	4	12	7
Sand equivalent	—	—	—	—	—	—	28	29	18	17	24	20	20	31
Compaction (Calif. impact)														
Maximum density (pcf)	—	129.8	134.0	132.3	135.9	131.0	139.8	139.8	135.4	151.0	144.1	138.5	136.0	143.5
Optimum moisture (percent)	—	10.4	10.0	8.7	7.5	6.8	7.4	6.6	8.3	4.8	6.0	8.8	9.2	6.3
Strength data														
Conventional test[a] Cohesion (psi)	—	42	0	35	69	83	43	45	35	42	45	69 42	63 42	45
Friction angle (deg)	—	4	17	25.5	17	7.5	24	31	27	11	28	10 25	9 20	24
Multiple stage test	—	—	—	—	—	—	—					— —	— —	
Cohesion (psi)								29	28	42	31			37
Friction angle (deg)								33	29	12	30			27

[a]Average values of all specimens tested in sample.

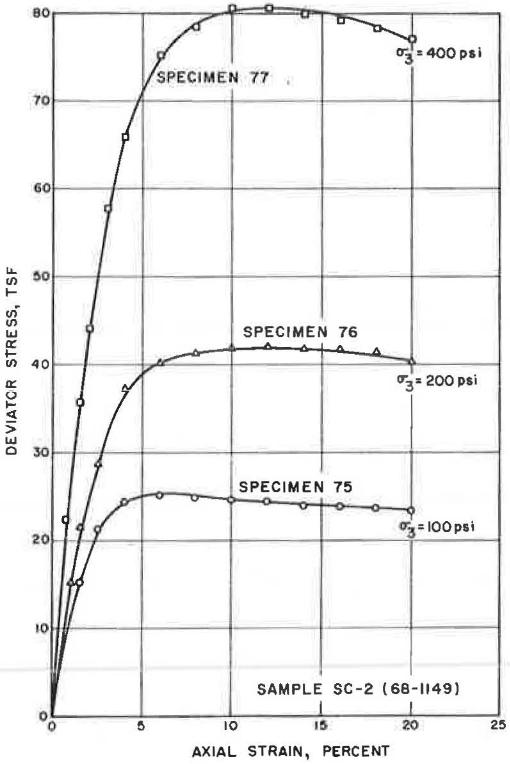


Figure 5. Stress-strain curves from conventional triaxial tests, typical Zone C material.

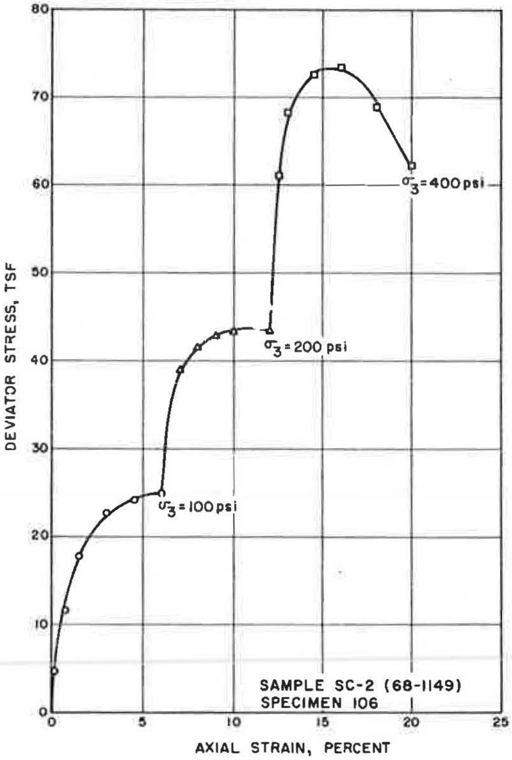


Figure 6. Stress-strain curves from multiple stage triaxial test, typical Zone C material.

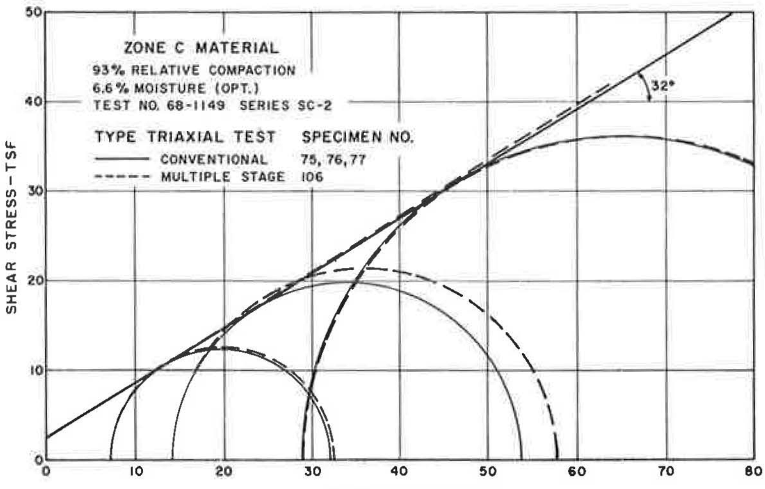


Figure 7. Comparison of Mohr's envelopes obtained with conventional and multiple stage triaxial tests, typical Zone C material.

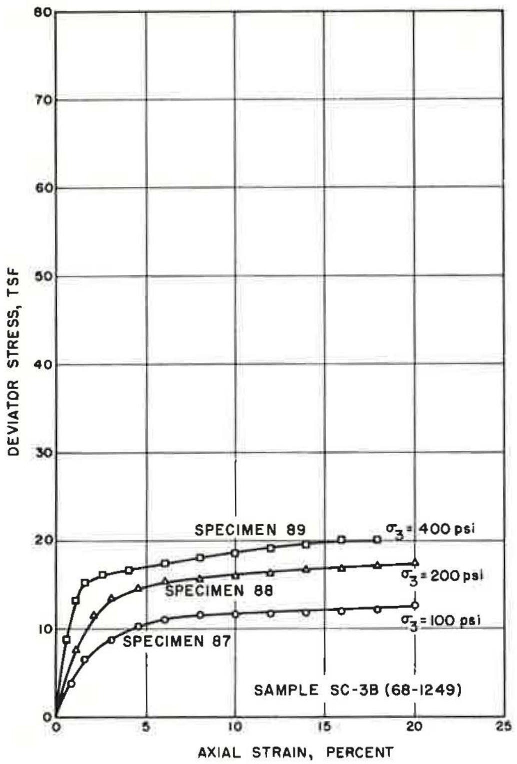


Figure 8. Stress-strain curves from conventional triaxial tests, weakest Zone A material.

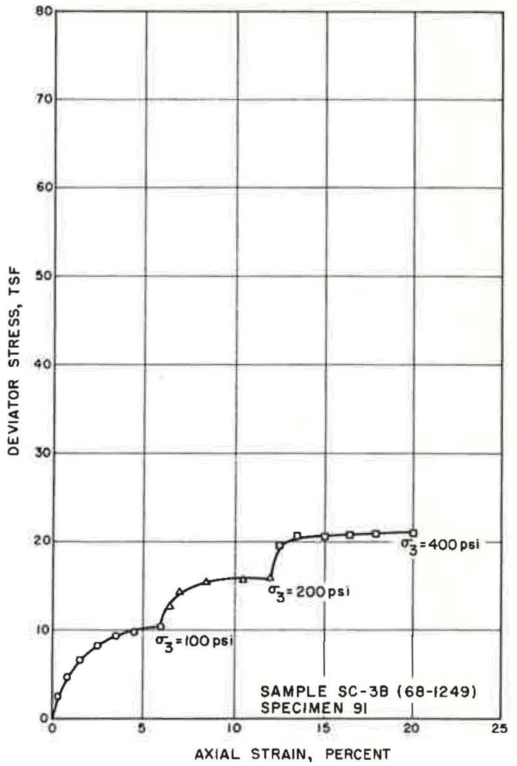


Figure 9. Stress-strain curves from multiple stage triaxial test, weakest Zone A material.

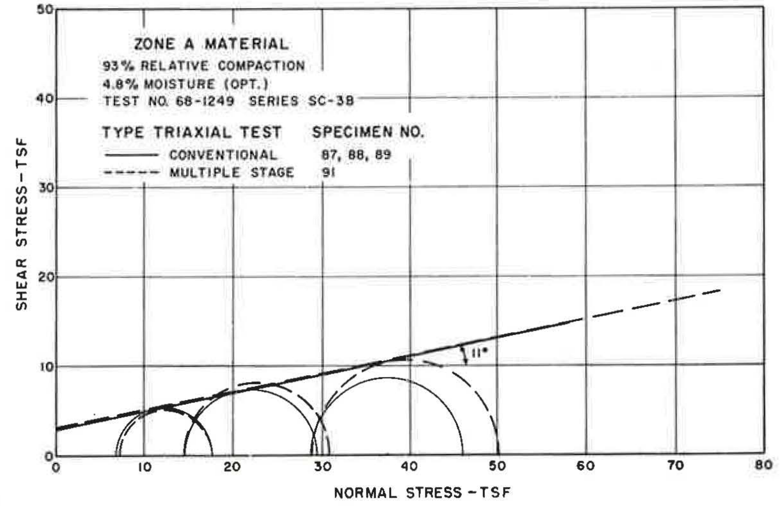


Figure 10. Comparison of Mohr's envelopes obtained with conventional and multiple stage triaxial tests, weakest Zone A material.

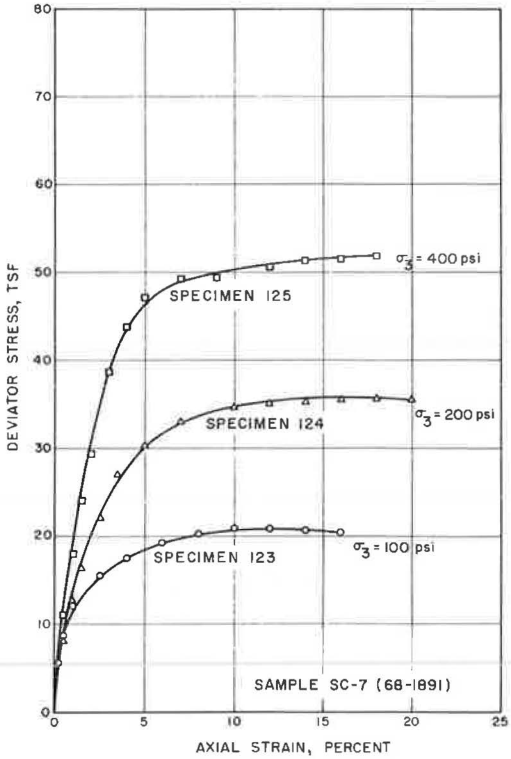


Figure 11. Stress-strain curves from conventional triaxial tests, typical Zone A material.

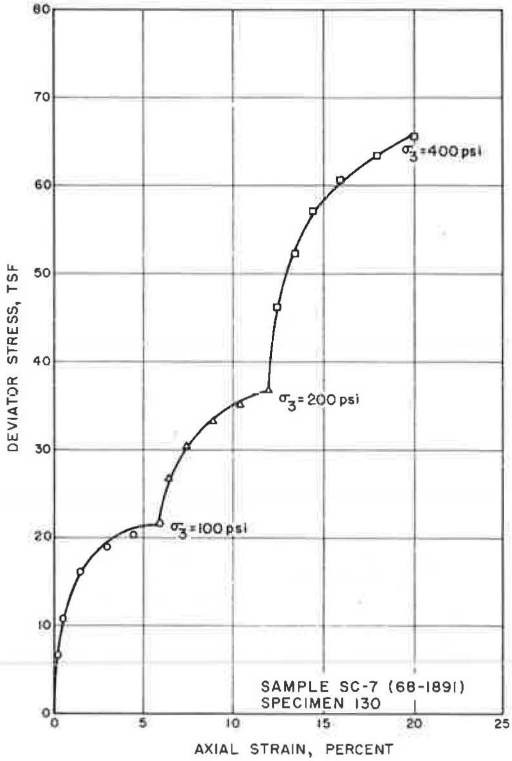


Figure 12. Stress-strain curves from multiple stage triaxial test, typical Zone A material.

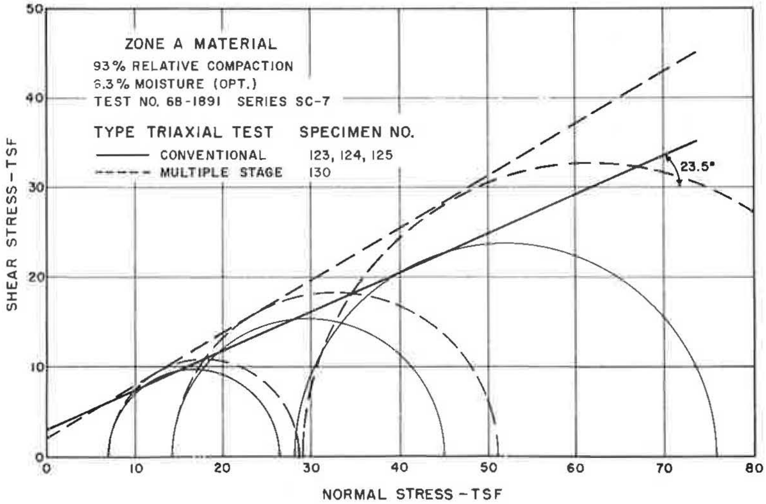
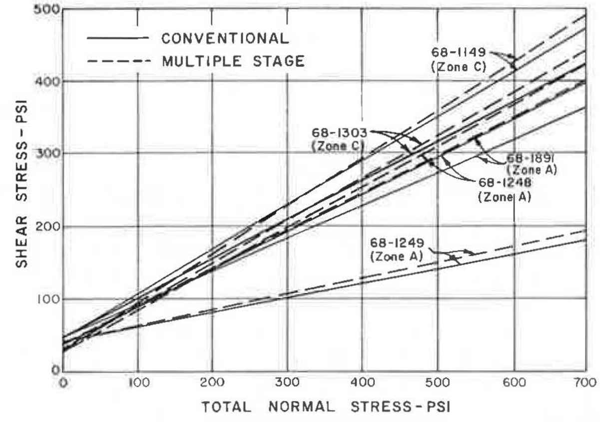
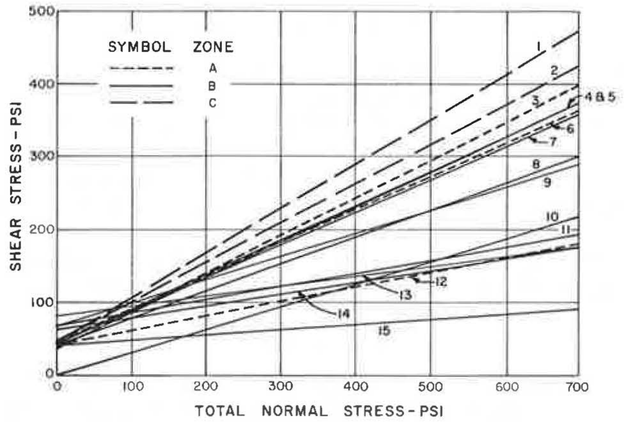


Figure 13. Comparison of Mohr's envelopes obtained with conventional and multiple stage triaxial tests, typical Zone A material.



TYPE TEST	COHESION (PSI)					ANGLE OF INTERNAL FRICT. (Φ)				
	68-1149	68-1248	68-1249	68-1303	68-1891	68-1149	68-1248	68-1249	68-1303	68-1891
CONVENT.	45	35	42	45	45	31°	27°	11°	28°	24°
MULTIPLE STAGE	29	28	42	31	37	33°	29°	12°	30°	27°

Figure 14. Comparison of Mohr's envelopes obtained with conventional and multiple stage triaxial tests.



ENVELOPE	ZONE	C (PSI)	Φ (DEG.)	TEST NO.
1	C	45	31	68-1149
2	C	45	28	68-1303
3	A	35	27	68-1248
4	B	42	25	68-1792
5	B	35	25.5	E
6	A	45	24	68-1891
7	B	43	24	67-2946
8	B	42	20	68-1842
9	B	69	17	F
10	B	0	17	C
11	B	69	10	68-1792
12	A	42	11	68-1249
13	B	83	7.5	5
14	B	63	9	68-1842
15	B	42	4	B

Figure 15. Comparison of Mohr's envelopes of materials sampled from different construction zones (conventional triaxial tests).

SUMMARY

The design studies for this project have been described (1). Construction of the project was accomplished during 1966 to 1968 using the controls established during design.

No serious problems were encountered during construction, and the facility has performed in a highly satisfactory manner since construction. A satisfactory installation for groundwater control was incorporated in the project. Rather large quantities of water have been removed by this installation. From observation it appears that hydrostatic pressures have not developed and groundwater has not constituted a serious problem.

During construction, various tests, including triaxial compression tests, were made to compare the design strengths with actual strengths being achieved during construction. Generally, the strengths achieved during construction were higher than had been predicted, based on tests made during design. Hence, some relaxation in the quality of material incorporated in the various zones was permitted. Selection of the material to be used in the various zones was based on visual observations, and changes were made by using material that appeared to be of slightly poorer quality than was originally intended for the various zones.

A novel testing technique was used for some of the triaxial compression tests. Rather than perform a test at a single lateral pressure for each specimen, tests were made using three different lateral pressures on a single specimen. The compressive load was increased to a predetermined strain or until evidence of incipient failure occurred. The confining load was then increased to the next increment, and the procedure repeated. Results of these tests indicate that, for this material and for the testing conditions used, this testing procedure produced results comparable to the normal test method where a single confining load is used for each specimen. The savings to be accrued by this alternate test method may prove to be substantial on future projects.

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The opinions, findings, and conclusions expressed in this report are those of the authors and not necessarily those held by the Federal Highway Administration.

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