SPECIAL TESTS FOR DESIGN OF HIGH EARTH EMBANKMENTS ON US-101

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New highway routes through the mountains of California will necessitate the construction of several embankments approaching 400 ft in height. The testing for use in design of high embankments involving soil-rock mixtures cannot be accommodated in conventional triaxial equipment. In an attempt to simulate embankment stress conditions on materials that were of the same types and sizes as contemplated for the prototype, large-scale triaxial tests were conducted on 12-in. diameter by 28-in. high specimens of minus 3-in. material at confining pressures up to 125 psi. Higher confining pressures to 400 psi were also used but these specimens were approximately 6 in. in diameter by 14 in. high, and the maximum particle size was 1½ in. Tests were conducted to obtain both total and effective stresses. Degradation of the materials under compaction, consolidation, and shearing was determined. The test data acquired have been used in design of one of the embankments with an overall height of 383 ft. The construction of this embankment was begun in June 1966 and was scheduled for completion in September 1968.

•DURING the next few years it will be necessary to relocate many miles of US-101 in northern California. This route is slightly inland from the Pacific Ocean and follows along Rattlesnake Creek and the Eel River. The terrain is very steep and much of the existing road is a two-lane road with rather sharp curves and low design speeds. The new route will be a four-lane highway with design speeds of the order of 60 mph.

To achieve these design standards, it will be necessary to construct at least one embankment that will be considerably higher than has been used in the construction of California highways. At least three creek crossings will have embankments in excess of 250 ft high. As an alternative it would be possible to build structures at these locations, but the cost of the structures would be considerably greater than the cost of embankments. Because of the proximity of the new road to Rattlesnake Creek and to the Eel River, it will be necessary to use slopes no flatter than 2:1 (2 horizontal to 1 vertical) and preferably as steep as 1½:1 in order to avoid encroachment of the toe of the embankment into the nearby creek or river.

It was deemed necessary that studies should be made of the strength characteristics of the embankment material. Normally, highway embankments in California have been constructed of roadway excavation with 1½:1 or 2:1 slopes and heights as great as 100 to 200 ft. It was decided to undertake a rather comprehensive testing program using typical materials that would be available from roadway excavation along the projects to be constructed.

SAMPLES

The materials in this area are derived from the large belt of sedimentary rocks of Upper Jurassic to Cretaceous age. The sedimentary rocks consist chiefly of coarse sandstones, shales, and minor conglomerates. A negligible amount of volcanic rocks

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is interbedded with sandstone and shale at a few places along the proposed alignment. Basalt, greenstone, chert, and limestone are also present in minor quantities.

Three sites were selected for securing samples believed to be representative of the material that would be encountered in construction. A sample of material was secured from each of these sites and consisted of approximately 3 cu yd of material per site. The material was excavated with a small shovel or loader and transported from the sites to the laboratory in Sausalito, a distance of approximately 185 miles. In normal operations the material from these roadway cuts will be excavated by the use of loaders or rippers, with occasional blasting necessary in the zones of harder sandstone.

One of the samples, probably the one of the poorest quality, was primarily shale in a slightly weathered condition. The second sample consisted of somewhat weathered fractured sandstone with some shale. The third sample was primarily a combination of sandstone and shale with somewhat more severe weathering than was involved in the other two samples. The testing reported here involves only the third material. A fourth type of material will be available from roadway cuts in limited quantities. This material will consist of fairly fresh to weathered massive fractured sandstone. It was not felt that it was necessary to test this sandstone because its strength will be considerably higher than any of the three samples tested.

Four assumptions were made in formulating the testing program. First, it was assumed that the samples secured would be representative of the material that would be encountered in roadway excavation. Second, the materials would be handled in such a manner as to ensure a proper moisture placement range. Third, some selection of material would be possible, and hence the material could be zoned using the poorer quality in the areas where strength was not so critical and using the better quality where the higher strength was essential. Fourth, detrimental excess hydrostatic pressures would not develop if the material was placed somewhat dry of optimum.

TESTS

The principal parameters that were desired from the testing program performed by the U.S. Army Engineers Division Laboratory at Sausalito, California, were shear strengths obtained under varying molding moisture-density conditions. Further, these values were needed on materials containing appreciable quantities of large gravel sizes and under test conditions of high confining pressures.

In addition to the triaxial compression tests for strength determinations, permeability tests were performed on large specimens under varying confining pressures to determine the k value with change in void ratio as well as to observe the degree of degradation with change in confining pressure. The need for inclusion of permeability tests was immediately apparent when the magnitude of degradation was observed from the initial shear specimens. A study of degradation was made for the purpose of isolating the effect of compaction, consolidation, and shear, and may be noted in Figures 1 and 2. The percent compaction as noted in the various figures is a percent of the maximum as established from impact effort and is the Corps of Engineers Military Standard 621 CE55 (AASHO T180-57, Method D). For most soils, the California impact test, as performed according to Test Method No. Calif. 216, gives approximately the same values of optimum moisture and maximum density as the Corps of Engineers test.

Three types of triaxial compression tests were conducted. The majority of tests were the Q, or unconsolidated undrained type, in which the specimens were sheared at predetermined molding moistures and densities on application of the confining or lateral pressure and without admission of any water. A limited number of tests designated as R modified were made to detect the possible increase in strength from consolidation before shear. No additional water was added to these specimens to gain saturation. The conventional R, or consolidated undrained tests, were conducted on remolded specimens that had been saturated and consolidated before shearing. All three types of tests were conducted with closed systems in which no water was allowed to drain from the specimen during shearing action. Figures 1 and 2 show the field grading of the minus 3-in. and minus 1½-in. portions of the material. All specimens tested at lateral pressures between 15 and 125 psi contained minus 3-in. particle sizes. Between lateral pressures of 125 and 400 psi, the maximum particle size was minus 1½ in.

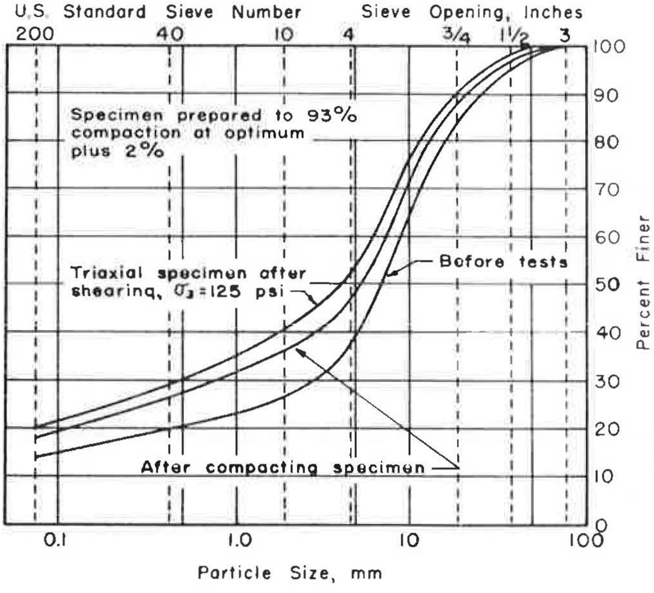

Figure 1. Particle size analysis of minus 3-in. material "before" and "after" tests.

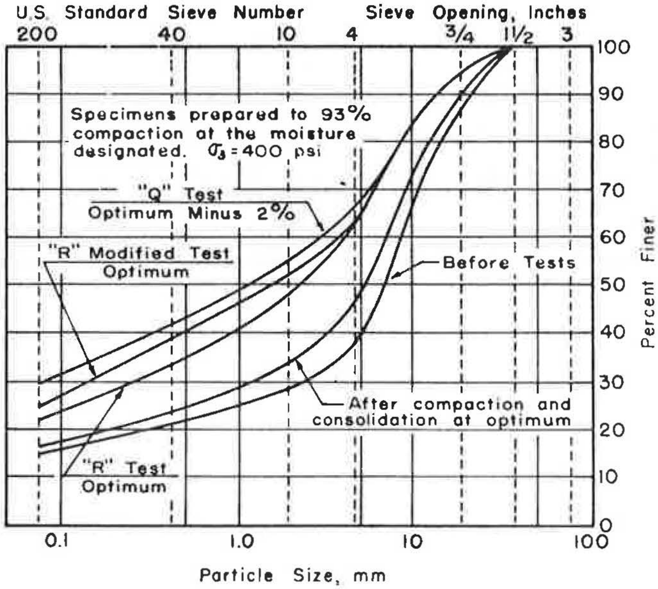

Figure 2. Particle size analysis of minus 1½-in. material "before" and "after" tests.

The specimen size for minus 3-in. material was 12-in. diameter by 27.6 in. high and the minus $1\frac{1}{2}$-in. material was tested in a 6-in. diameter by 13.8-in. high specimen. In the 12-in. diameter apparatus (2, 4), the pore pressure was measured at seven different locations along the central axis of the specimens. Five of the sensing points were within the specimen and one at either end. The minute coaxial cables to the transducers in the specimens were brought through the rubber membrane at the elevation of the transducer and a minimum of miniature cable was actually within the specimen. The cables were then lead through the base of the apparatus to a measuring console. The volume of each transducer was about 0.0015 cu ft and cylindrical in shape. In the apparatus used for the minus $1\frac{1}{2}$-in. material at the higher lateral pressures, the pore pressure was sensed only at the ends of the specimen (6). Figure 3 shows the apparatus that accommodated the 12-in. diameter specimens.

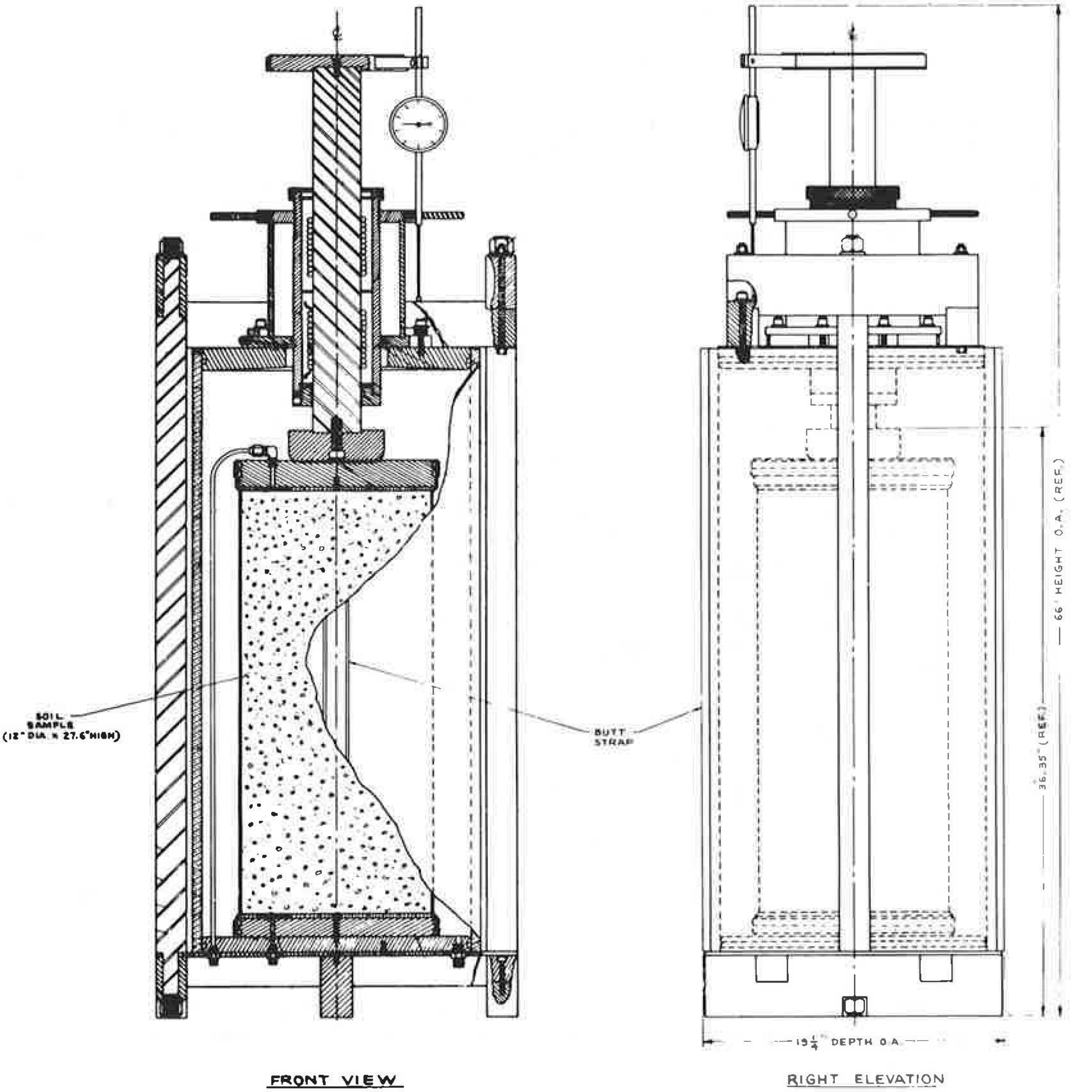

Figure 3. Large-scale triaxial apparatus.

All tests were performed under a controlled rate of strain. For the Q test, the rate was established by observing all of the pore pressure cells simultaneously to detect any development of a pressure gradient. The specimens were strained as fast as possible while still permitting the pore pressure to remain equal through the column of soil. This allowed strain rates varying between ¾ and 1 percent per minute. For all R tests, it was necessary to reduce the strain rate to about 0.2 percent per minute to eliminate the development of a pressure gradient. The test was stopped when both the maximum deviator stress and the maximum effective principal stress ratio had been reached. Volume changes during the shearing action were noted on pressure burettes connected to the water surrounding the specimen in the pressure chamber.

One of the principal difficulties experienced in large-scale and high-pressure triaxial testing is to fabricate membrances that will withstand high pressures and still bridge angular surface interstices without developing membrane restraints that are noticeably significant. On the specimens tested at the two highest lateral pressures in the 12-in. diameter apparatus, it was necessary to use double membrances. On several of the 6-in. diameter specimens, it was necessary to employ a series of overlapping polyethylene strips about 2½ in. wide placed vertically between the two membrances. The thickness of the polyethylene strips was varied with the angularity of the material and the depth of the surface interstices.

Figure 1 shows the degradation results of a large-scale triaxial test performed with a lateral pressure of 125 psi. The large degree of degradation resulting only from compaction is somewhat the exception and not the rule. In Figure 2 it will be noted that the largest degree of degradation was attained for the specimen compacted 2 percent dry of optimum. This is in direct relationship to the magnitude of energy expended during the molding operations because the drier the material, the more effort is expended to meet a specific density. To demonstrate that degradation cannot be predicted with any degree of accuracy in a material of this type, it may be noted that the R modified test that was performed at placement moisture shows a greater degradation than the standard R test that was performed on a saturated specimen. It is, of course, impossible to separate the degradation due to compaction from that resulting from consolidation. It would appear, however, from observing the results in Figure 2 that about 10 percent of the gravel sizes (plus No. 4) was reduced to minus No. 4 particles when the permeability test at the 400-psi confining pressure was made. The permeability for a specimen molded to 93 percent compaction and at optimum moisture was 1×10^{-2} cm/sec or about 30 ft per day. After consolidation with a confining pressure of 400 psi, the permeability was reduced to 1.3×10^{-3} or about 3.5 ft per day.

The data in Figure 4 are presented to show the change in pore pressure with variation in placement moisture at comparable confining pressures. The figure further serves to demonstrate the magnitude of pore pressure when compared to a specific confining pressure. The percent compaction was the singular factor common to all specimens. The high pore pressure value for the R test with a confining pressure of 400 psi is not unexpected as it represents the only test that was saturated before shearing. In comparing the R modified test at optimum moisture and chamber pressure of 400 psi with the comparable Q test, it may be observed that the pore pressure is somewhat higher in the Q test (Table 1). This is a result of an appreciably larger void ratio at failure, which resulted in a dissipation of the pore pressure because both specimens had the same placement moisture content.

The Mohr failure envelopes for total stress and effective stress values are shown in Figures 5 and 6. The letter designation for the various envelopes is common to both figures and the test conditions for each envelope are given in Table 2. The envelopes were developed from a minimum of three tests and a maximum of five tests at confining pressures ranging between 15 and 400 psi. As anticipated, the highest strength envelope from the total stress plot was developed on materials prepared 2 percent dry of optimum and to 93 percent compaction. The lowest strength was at the lowest density investigated (85 percent compaction) and 2 percent wet of optimum. This produced a range of ϕ of 14 to 34 deg. The angles of the effective Mohr failure envelopes (Fig. 6) bracket a much smaller range of 30 to 38 deg.

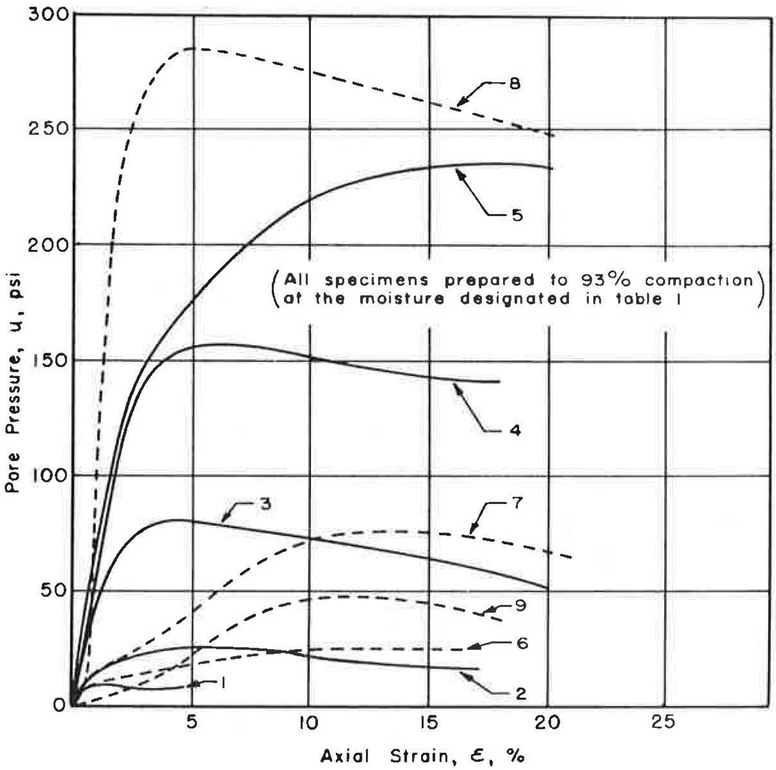

Figure 4. Plot of pore pressure and axial strain.

TABLE 1

TEST DATA FOR CURVES ON FIGURE 4

Curve No.	Type of Test	Lateral Pressure (psi)	Percent Compaction	Specimen Molding Moisture
1	Q	15	93	Optimum +2 percent
2	Q	60	93	Optimum +2 percent
3	Q	125	93	Optimum +2 percent
4	Q	200	93	Optimum +2 percent
5	Q	400	93	Optimum +2 percent
6	Q	400	93	Optimum -2 percent
7	Q	400	93	Optimum
8	R	400	93	Optimum
9	R (modified)	400	93	Optimum

TABLE 2

TEST DATA FOR FAILURE ENVELOPES ON FIGURES 5 AND 6

Envelope No.	Type of Test	Lateral Pressure (psi)	Percent Compaction	Specimen Molding Moisture
A	Q	400	85	Optimum +2 percent
B	Q	400	88	Optimum
C	Q	400	85	Optimum -3 percent
D	Q	400	93	Optimum +2 percent
E	Q	400	93	Optimum
F	Q	400	93	Optimum -2 percent
G	R (modified)	400	93	Optimum +2 percent
H	R (modified)	400	93	Optimum
I	R	400	93	Optimum

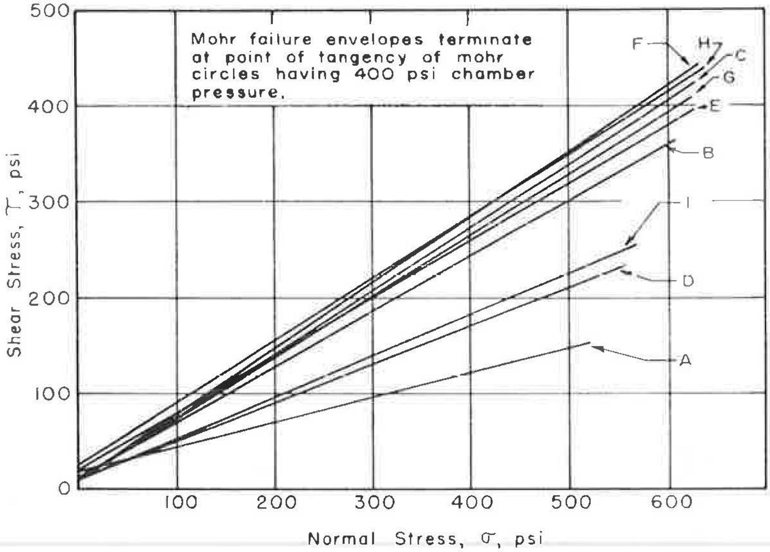

Figure 5. Failure envelopes for total stresses.

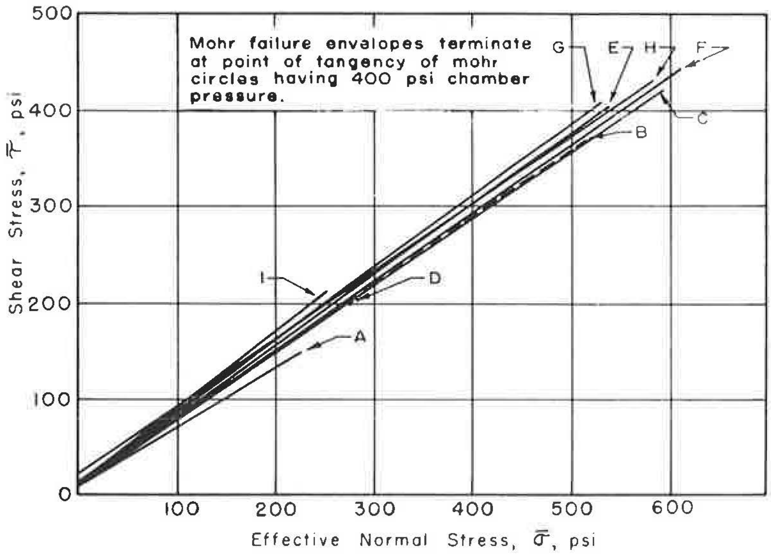

Figure 6. Failure envelopes for effective stresses.

DESIGN

In addition to this sampling and testing program, geologic studies were conducted and borings were made to further evaluate the material that would be available for construction of the 383-ft high embankments at Squaw Creek and to ensure that the testing program had included the available material. These studies consisted of geologic mapping, seismic surveys, and vertical and horizontal core and auger borings made in cuts in the vicinity of this embankment. From these data it was possible to determine roughly the percentages of the various types of material that would be available for embankment construction. The materials available could be classed in three general categories. The first category consisted of overburden and severely weathered material with a moderate to high percentage of clay as well as silt, sand, and gravel sizes. The second category consisted of somewhat weathered sandstone and shale of fair to good quality. The third category was primarily weathered and somewhat fractured and jointed sandstone.

Using the data available from this exploration and the testing program, a study was made of the possible zoning of materials in the embankment, and the stability of several typical sections was calculated. The following strength values, based on data from the testing program, were used in the various zones in the proposed section:

Zone	Cohesion (psf)	Angle of Friction (deg)	Percent Compaction
A	500	15	90
B	500	25	93
C (est. values)	0	35	95

Stability analyses were made with two computer programs that had been developed or modified for use on the IBM-704 electronic computer. It was evident from the borings and geologic investigation that there would be a shortage of the best quality material, that is, sandstone from the cuts; hence, the cross section of the sandstone portion of the proposed embankment was kept to a minimum that would still result in a stable embankment. The cross section for the embankment at Squaw Creek is shown in Figure 7.

Considerable selection of material from roadway excavation will be necessary to construct an embankment with the proposed cross section. This will necessitate long hauls and possibly stockpiling of material to ensure its placement at the proper location in the embankment.

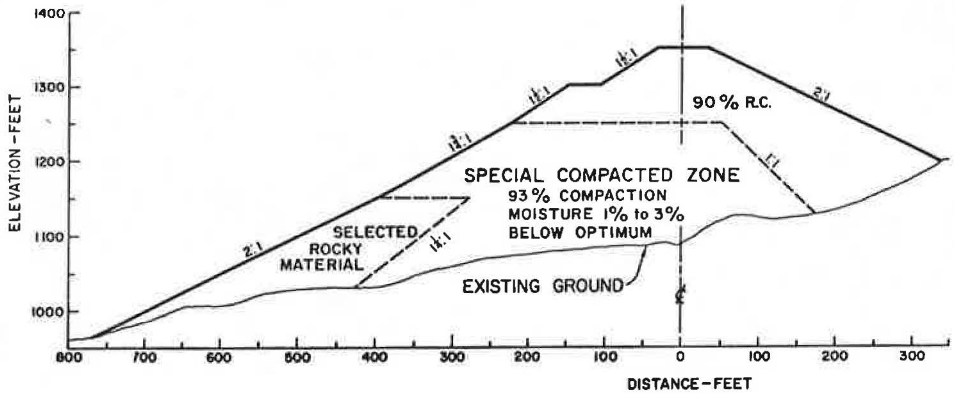

Figure 7. Typical section at Squaw Creek.

CONSTRUCTION

A $6,000,000 contract for construction of the road in this area was awarded in June 1966 that included construction of an embankment with a maximum height of 383 ft across Squaw Creek. This embankment requires approximately $2\frac{1}{2}$ million of a total of approximately 3 million cu yd on the project. An aerial view of a portion of the project is shown in Figure 8 after the clearing and grubbing operations had been completed and a stabilization trench had been constructed in the bottom of Squaw Creek.

The specifications require the placement of selected material in Zones C and B. More rigid control of moisture content and higher relative compaction than is normally required is specified for construction of Zone B. Zone C consists of selected material and is the sounder and larger rock encountered in excavation of designated cuts. Likewise, the material for Zone B consists of selected rocky, granular material from designated cuts, and does not contain material that is normally classified as overburden nor does it contain an appreciable amount of shale, clay, soil, or vegetable matter. Zone B material is being compacted to a relative compaction of at least 93 percent, in lieu of the standard requirement of 90 percent, and at a moisture content of 1 to 3 percent less than optimum moisture as determined by the California impact test. (Relative compaction is defined as the ratio of the in-place density of a soil or aggregate to the test maximum density of the same soil or aggregate when compacted by a specific test method.)

This project is one of several on which nuclear gages are being used to determine relative compaction and in-place moisture of embankment material. In-place density is being determined by nuclear Test Method No. Calif. T-231 instead of the normal sand-volume method, which is part of Test Method No. Calif. 216.

The aerial view shown in Figure 8 reveals the ruggedness of the terrain through which this project passes. Most of the material is quite wet because of high groundwater table and springs. The average annual rainfall is about 70 in. and most of it falls between October and April. The existing road is well-known for its past history of

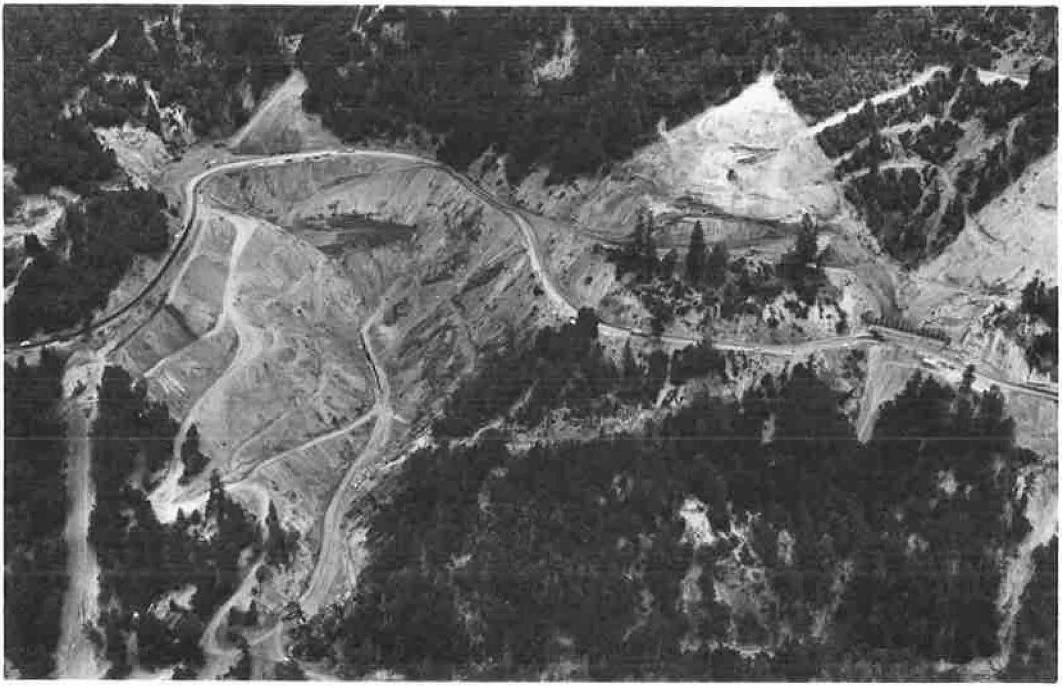

Figure 8. Aerial view after clearing and grubbing and stabilization trench construction at Squaw Creek.

landslides. Underdrains are being installed in all cut sections. Unsuitable material has been stripped from the embankment foundations and a 3-ft layer of permeable material placed beneath the embankments where needed. Supplemental funds in the amount of $350,000 have been provided for possible slide removal and additional drainage work.

The time schedule provides for 300 working days with two winter shutdowns of approximately six months each. The estimated completion date was September 30, 1968. It is recognized that the time limit is insufficient for the contractor to accomplish the work by working the normal number of hours per day or week on a single-shift basis. The specifications require that additional shifts may be necessary to ensure that the progress of the work conforms to the progress schedule.

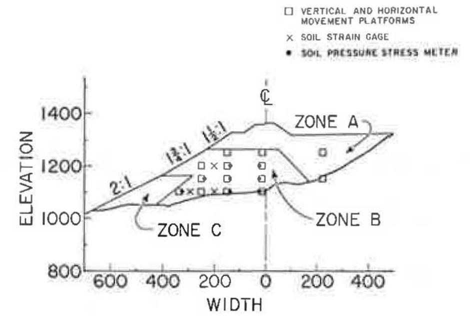

Figure 9. Typical section at Squaw Creek showing location of construction control devices.

To observe the behavior of the embankment during and subsequent to construction, the embankment is being instrumented with settlement platforms, horizontal movement plates, strain gages, stress meters, and piezometers. These devices will warn the resident engineer of changes in conditions as the embankment is constructed. The locations of these devices are shown in Figure 9. The first level of devices near the bottom of Zone B were installed during September 1966.

Samples of the uncompacted material and compacted "undisturbed" samples from Zone B will be secured to perform other series of triaxial tests to check on the original design of the embankment.

This paper demonstrates the need for correlating sampling, exploration, and testing of a combination of soil and rock to a specific problem in engineering design and construction. It is believed that the information obtained from the types of special apparatus used and from the magnitude of the confining pressures employed will be of immediate value to other professional people in the field of soil mechanics.

ACKNOWLEDGMENTS

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