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REVIEW OF SHEARING STRENGTH OF ROCKFILL

By Thomas M. Leps,[1] F. ASCE

INTRODUCTION

The shearing strength of rockfill for dams and dikes has been a subject of interest, concern and speculation among civil engineers for at least 120 yr. As noted by I. C. Steele (8), "the design of rockfill dams originated in California soon after the discovery there of gold in 1848. They were . . . generally of small proportions daringly designed." From the standpoint of engineering knowledge of the strength of the basic material used in such dams, it could almost be said that the design of rockfill dams continued to be daring for many decades, perhaps well into the 1940's. Actually, such was not the case since design was not based on diagnostic testing of the strength of rockfill, but on the satisfactory performance of many prototype fills, together with the application of the always essential ingredients—broad reasoning and engineering intuition. The use of the latter, more or less empiric approach to design has been unusually satisfactory over the years, to the extent that it cannot be said at this time that its use has led to the loss of one rockfill dam. On the other hand, it can reasonably be said that most rockfill designs that have been prepared in the past 50 yr have been built without the designer's certain knowledge as to how safe, or daring, his design actually was, in terms of current understanding of the significance of factors of safety for embankments. This inability to quantify the degree of stability of a rockfill dam has been a growing challenge to civil engineers. As dam heights have increased, and the consequences of a failure have multiplied enormously as a result of population growth and occupancy of the areas below dams, the challenge to know and understand rockfill behavior has progressed beyond satisfaction with the empiric approach to an approach which incorporates studying the stress-strain behavior of rockfill under compressive and shearing loads. Such an approach has been analogous to that utilized in the analysis of earthfill dams.

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[1] Consulting Civ. Engr., Atherton, Calif.

It has lagged far behind the development in soil mechanics of knowledge of the stress-strain behavior of soils, principally because of the great cost of building testing equipment large enough to handle prototype-sized pieces of rock, but also because the number of rockfill dams being built each year has been relatively small.

EARLY DEVELOPMENTS

In the past 20 yr, the major efforts in shear testing rockfill have been made through use of the triaxial shear type of testing apparatus which tests a cylindrical shaped specimen enclosed in a rubber sheath. This type of apparatus was first developed for soil testing by Leo Jurgenson (3) at MIT in 1933 to 1934. Jurgenson's model was extensively modified and improved for use on small diameter (1.5 in. to 3 in.) soil specimens between 1936 and 1940, separately by H. A. Fidler at MIT and by J. D. Watson at Harvard, working under the close guidance, respectively, Donald W. Taylor and Arthur Casagrande. Concurrently, Cecil Morris, under the guidance of Loyd W. Hamilton, was developing comparable, but somewhat larger, triaxial soil testing apparatus at the U.S. Bureau of Reclamation's Denver earth dams laboratory, with emphasis on instrumentation to detect and measure pore pressures in the test specimens.

From the three foregoing simultaneously progressing programs emerged the present generation of triaxial shear testing apparatus. In 1940, however, no one was attempting consciously to test materials in such equipment if the maximum grain size of the materials exceeded about 1/4 in. Obviously, such materials could not be assumed to have characteristics comparable to those of rockfill.

In the post-World War II period, however, the pressure to learn in the laboratory about rockfill behavior was spurred by acceleration in the pace of dam investigations and construction. The logical vehicle at that time for studying such behavior was thought by many to be an adaptation of the soil mechanics triaxial test equipment. The first individual to construct a triaxial test apparatus for rockfill was Earl B. Hall, then working in the soils laboratory of the Corps of Engineers, South Pacific Division, at Los Angeles. The dam which justified the apparatus and the testing program was Isabella Dam on the Kern River in California. The dam was intended to be a combination earth and rockfill embankment, with the rockfill to be made of quarried granite. Hall devised an apparatus for testing 18-in.-diam by 36-in.-high specimens. The specimen was enclosed in a cylindrical rubber membrane and was loaded by partially evacuating the internal air pressure. It was tested for compressive strength by loading it axially in a standard, concrete cylinder testing machine. For Isabella Dam, Hall tested rock fragments smaller than 4 in. in size but larger than 1 in. It is believed that this equipment represented the first attempt in the U.S., and possibly anywhere, to test rockfill-sized material in triaxial shear.

TEST EQUIPMENT

Subsequent to Hall's 1947-48 equipment development and use, enthusiasm for constructing large scale triaxial testing equipment for rockfill studies

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GeoTesting, Calif.
Soil Mechanics and Foundations Engineers, Calif.
United States Bure of Reclamation, Denver, Colo.
Infiernillo, Mexico
Infiernillo, Mexico
UCB, Richmond, Calif.

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grew, but the heavy costs involved have limited the number of actual installations. As of this time (1969), the writer knows of several in the Western Hemisphere (see Table 1).

TABLE 1.—EQUIPMENT DEVELOPMENT AND USE

Location (1)	Specimen diameter, in inches (2)
El Infiernillo Dam, Mexico	44.5
El Infiernillo Dam, Mexico	8
University of California, Richmond, California	36
University of California, Richmond, California	12
Corps of Engineers Laboratory, Marietta, Georgia	15
Corps of Engineers Laboratory, Sausalito, California	12
Corps of Engineers Laboratory, Portland, Oregon	12
GeoTesting Inc., San Rafael, California	12
Soil Mechanics and Foundation Engineers, Inc., Palo Alto, California	12
USBR Lab., Denver, Colorado	9

TABLE 2.—LARGE SCALE TRIAXIAL TESTING DEVICES

Location (1)	Maximum lateral pressure, in pounds per square inch (2)	Specimen diameter, in inches (3)	Maximum stone size, in inches (4)
Corps of Engineers Sausalito, Calif.	125	12	3
Corps of Engineers Sausalito, Calif.	1,500	6	1.5
GeoTesting, Calif.	550	12	3
GeoTesting, Calif.	2,000	6	1.5
Soil Mechanics and Foundations Engineers, Calif.	750	12	3
United States Bureau of Reclamation, Denver, Colo.	100	9	3
Infiernillo, Mexico	350	44.5	8
Infiernillo, Mexico	700	8	1.5
UCB, Richmond, Calif.	750	36	6

To accommodate the need for stress-strain data on rockfill under the high loadings contemplated in the higher and higher rockfill dams being planned, the foregoing testing devices were designed to test at the highest practicable lateral pressures. According to published data, Table 2 applies to the various large scale triaxial test devices.

BACKGROUND OF REVIEW

While recognizing the justified interest in and importance of studying the performance characteristics of rockfill at high lateral pressures, as reported in recent years (1,2,7), the writer wishes also to point out and review the importance of those characteristics at low confining pressures. "Low" pressures, as used in the following analysis, are considered to be generally in the range of from 0 psi to 10 psi lateral pressure. To emphasize the importance of this very low range, some authorities believe that the critical sliding surface in stability analyses of rockfill slopes on very competent foundations, whether analyzed by the infinite slope method or by the sliding circle or wedge method, is very shallow and generally involves confining pressures in the 0 to 10 psi range. Furthermore, it is becoming increasingly clear that many of the "daringly designed" rockfill dams of the 19th century could not possibly have stood for the past 70 yr to 100 yr unless the rockfill used was actually stronger than has been indicated by triaxial tests in the 20 psi to 200 psi lateral pressure range. For these reasons, it seems important to understand the stress-strain performance of rockfill at low pressures to the same degree as at intermediate and high pressures, and especially so in the light of representative quantitative findings of authorities such as Marsal, et al. (7), who have reported that the rockfill for El Infiernillo Dam has been found to have friction angles varying from 50° at 5.7 psi lateral pressure to 34° at 355 psi. Stated in another manner, this remarkable finding means that the fill is 76 % stronger per unit of confining pressure at the lower pressure than it is at the higher one. Such a large variation leads one to the inescapable conclusion that it may be a gross oversimplification in stability studies to talk in terms of a single "average" friction angle for a given rockfill. It also raises the intriguing and practical question of what the variation of the friction angle of rockfill at confining pressures less than 5.7 psi is.

ROCKFILL STRENGTH CHARACTERISTICS

To review this question to the extent possible at the present time, the writer undertook to assemble and analyze published data readily available for individual large scale triaxial tests on gravels and rockfill. These data included tests by Hall dating back to 1948 for the Corps of Engineers and later at GeoTesting, tests by Holtz and Gibbs for the U.S. Bureau of Reclamation published in 1956, and tests by Marsal and his associates at El Infiernillo published in 1965 and 1966. Both to make the data from the several sources easy to compare and to simplify their possible application to stability analyses, the shear test results on each specimen for which data have been reported have been recomputed where necessary to show friction angles as a function of normal pressures across the failure plane, as deduced from use of the Mohr diagram. The data from all tests on reasonably representative rockfill materials, including a range from gravel specimens to -8-in. rock specimens, and covering a variety of 15 different rockfill sources, are plotted in Fig. 1. The backup data for this plot are tabulated in Table 3. For comparison, Fig.

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Location (1)	M
Isabella	Gra
Cachuma	Gra
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Cachuma	Qua
Cachuma	Qua
Oroville	Tail
Soledad	Grav
Infiernillo	Dior
Infiernillo	Dior

TABLE 3.—FRICTION ANGLE OF ROCKFILL

Location (1)	Material (2)	Maximum particle, in inches (3)	Dry density, in pounds per cubic foot (4)	Normal pressure, in pounds per square inch (5)	Maximum friction angle, in degrees (6)
Isabella	Granite	4	97.0	7.5	47.0
		4	95.0	23.0	43.5
Cachuma	Gravel	0.75	126.0	6.3	54.7
		0.75	125.0	11.3	49.5
		0.75	123.0	21.5	44.5
		0.75	125.0	43.0	45.0
		0.75	124.0	84.0	41.0
		0.75	124.0	165.0	39.5
		0.75	125.0	162.0	38.5
Cachuma	Gravel	3	126.0	6.2	54.0
		3	124.0	11.8	49.5
		3	124.0	22.1	47.0
		3	127.0	45.0	46.5
		3	125.0	86.0	43.5
		3	127.0	167.0	41.5
Cachuma	Quartz Monz.	3	122.0	20.0	40.0
		3	123.0	65.0	39.5
		3	122.0	123.0	39.0
		3	129.0	22.0	44.0
		3	128.0	60.0	42.0
		3	130.0	125.0	41.0
Cachuma	Quartz Monz.	3	117.0	42.0	44.0
		3	117.0	59.0	41.0
		3	127.0	44.0	47.0
		3	127.0	63.0	46.0
Oroville	Tailings	1.5	144.0	490.0	40.0
		1.5	142.0	484.0	38.8
		1.5	148.0	424.0	43.0
		1.5	147.0	700.0	40.5
		1.5	148.0	1160.0	40.0
		3	143.0	208.0	42.0
		3	143.0	206.0	41.3
		3	150.0	213.0	45.0
Soledad	Gravel	3.5	1.02[a]	8.2	44.8
		3.5	0.88	16.6	42.8
		3.5	0.64	8.8	50.0
		3.5	0.69	16.9	47.2
Infiernillo	Diorite	7	0.82	8.3	44.0
		7	0.86	16.1	44.0
		7	0.69	9.1	49.5
		7	0.70	17.2	46.5
		7	0.65	8.7	49.0
		7	0.70	11.7	46.5
		7	0.60	16.0	46.5
Infiernillo	Diorite	8	0.45	9.8	50.0
		8	0.61	21.4	46.1
		8	0.62	44.5	44.4
		8	0.73	114.0	40.7
		8	0.55	230.0	38.0
		8	0.51	385.0	35.0
		8	0.50	567.0	34.7

TABLE 3.—*CONTINUED*

Location (1)	Material (2)	Maximum particle, in inches (3)	Dry density, in pounds per cubic foot (4)	Normal pressure, in pounds per square inch (5)	Maximum friction angle, in degrees (6)
Infiernillo	Conglomerate	8	0.62[a]	16.1	46.1
		8	0.55	45.0	45.5
		8	0.55	113.0	41.0
		8	0.62	230.0	39.4
		8	0.51	390.0	37.8
		8	0.45	570.0	37.1
		8	0.51	44.8	45.6
		8	0.50	114.0	42.2
		8	0.40	232.0	39.5
		8	0.40	390.0	37.6
		8	0.46	570.0	36.3
Malpaso	Conglomerate	8	0.42	9.8	50.0
		8	0.35	21.9	49.2
		8	0.42	45.5	48.0
		8	0.32	116.0	45.2
		8	0.44	230.0	39.0
		8	0.38	390.0	39.0
		8	0.40	570.0	36.9
		8	0.43	570.0	37.2
		8	0.42	570.0	37.4
		8	0.33	570.0	38.9
		8	0.33	575.0	39.5
Pinzandaran	Gravel	8	0.33	10.2	53.1
		8	0.36	22.3	52.3
		8	0.32	45.6	48.5
		8	0.32	116.0	45.5
		8	0.32	233.0	42.5
		8	0.34	390.0	39.3
		8	0.35	573.0	38.9
Infiernillo	Basalt	7	0.30	10.4	60.0
		7	0.30	25.6	55.0
		7	0.30	122.0	45.7
		7	0.30	239.0	42.7
Infiernillo	Gneiss X	7	0.32	10.0	51.0
		7	0.32	23.0	45.0
Infiernillo	Gneiss Y	7	0.62	6.3	45.0
		7	0.62	20.4	41.3
Contreras	Gravel	7	0.68	8.0	41.6
		7	0.65	8.4	41.6
		7	0.68	17.3	41.0
		7	0.54	8.4	45.5
		7	0.53	17.0	45.5
Santa Fe	Andesite	7	1.06	8.0	42.7
		7	1.07	15.9	40.2
		7	0.92	8.7	48.0
		7	0.84	17.0	46.8
Fort Peck	Sand	No. 20	0.70	27.8	37.9
		No. 20	0.70	55.5	37.1
		No. 20	0.70	83.0	36.3
		No. 20	0.70	111.0	35.3

Location (1)	
Scituate	San
Ottawa Std.	San

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41.6
41.0
45.5
45.5
42.7
40.2
48.0
46.8
37.9
37.1
36.3
35.3

TABLE 3.—*CONTINUED*

Location (1)	Material (2)	Maximum particle, in inches (3)	Dry density, in pounds per cubic foot (4)	Normal pressure, in pounds per square inch (5)	Maximum friction angle, in degrees (6)
Scituate	Sand	No. 8	0.57[a]	27.8	38.0
		No. 8	0.57	55.5	37.5
		No. 8	0.57	111.0	35.5
Ottawa Std.	Sand	No. 14	0.59	6.9	33.6
		No. 14	0.59	13.9	33.0
		No. 14	0.59	27.8	31.8
		No. 14	0.59	41.6	30.8
		No. 14	0.59	55.5	30.0

[a] All subsequent numbers in this column are void ratios.

1 also shows data obtained by the writer (4,5,9) in 1938-39 in a research program on the shearing strength of sands, with Ottawa Standard Sand showing the lowest shear strength.

The tests shown in Fig. 1 represent an expenditure of probably well over a $1,000,000 in equipment and salaries. There undoubtedly are more test data which could usefully be added to the chart, but it is believed that the scope which has been recorded thereon is adequate to give any engineer who is experienced with rockfill design and stability problems an improved basis for evaluating the safety and stability of most rockfill embankments, provided he has specific knowledge of the general rock quality and method of fill construction.

In review, it is suggested that Fig. 1 has the virtues of: (1) Presenting a good overall perspective of the current understanding of the relation of friction angle to normal pressure in rockfills; and (2) illustrating the relative dearth of information at normal pressures below 10 psi. On the other hand, Fig. 1 has the shortcomings that:

1. It only roughly indicates the effects of relative density.
2. It only roughly indicates the effects of gradation of the rockfill.
3. The effect of crushing strength of the dominant sized rock particles is only vaguely suggested.
4. It gives no clue as to the influence of particle shape of the dominant rock particles.
5. It offers no evaluation of the influence of degree of saturation of the rock particles.

In regard to all such shortcomings, it would appear that the test data that are available are sufficient only to indicate trends. In this vein, it seems fair from a review of such data to make the following tentative statements.

Relative Density.—At a given normal pressure, increasing relative density results in an increased friction angle. Marsal's data indicates the maximum effect may be in the order of 3° to 4° at a normal pressure of 10 psi, declining to 1.5° at 500 psi. The USBR data indicate a similar effect.

Gradation.—Improving the gradation of rockfill, providing it is not done with fines, is found to increase the friction angle at any given normal pressure. At about 100 psi normal pressure, the USBR found that increasing the gravel content of a -0.75-in. gravelly sand from 20 % to 50 % increased the friction angle about 3.5°; and a -3-in. rock sample showed a 2° increase when rock was increased to 81 % from 65 %.

Particle Crushing Strength.—In their careful studies of the crushing of particles during large triaxial shear tests, Marsal et al. (6) concluded that particle breakage is a function of the mean intensity of particle contact forces and

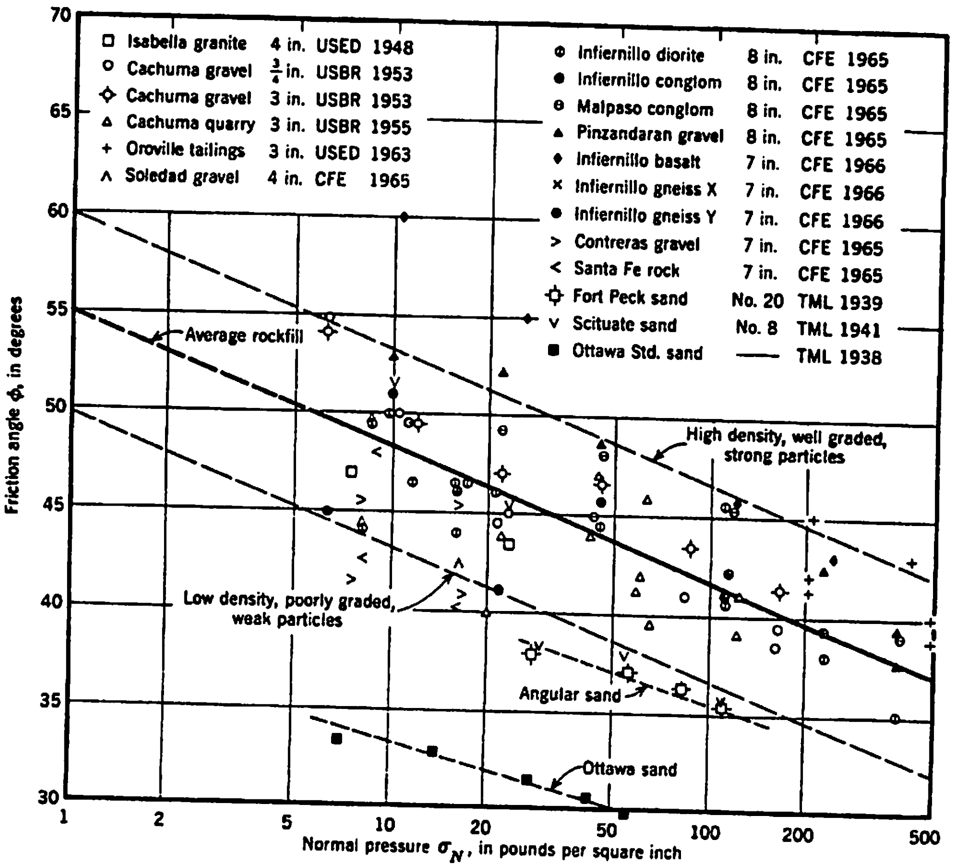

FIG. 1.—SHEARING STRENGTH OF ROCKFILL FROM LARGE TRIAXIAL TESTS

of the unconfined compressive strength of the rock particles. For the case where he has found minimum particle breakdown, for Pinzandaran Gravel, he found that 4.5 cm diam. cores had a strength of about 25,000 psi. Based somewhat on this finding, together with experience from other projects, the writer suggests that the following broad classifications may be meaningful when referring to Fig. 1: (1) Weak rock particles, of 500 psi to 2,500 psi strength; (2) average rock particles, 2,500 psi to 10,000 psi; and (3) strong rock particles, 10,000 psi to 30,000 psi.

Particle Shape.—There seems to be general agreement that, with all other factors remaining constant, the more angular the particles, the stronger will be the material. The USBR reported one comparison of -3 in., rounded, river gravels with -3 in., quarried rock in which they found the quarry rock to have a larger friction angle by about 2.5°. Marsal reports that -7 in., clean, hard, angular, quarried basalt for Infiernillo, at normal pressures in the order of 10 psi to 20 psi, possessed friction angles as much as 10° to 15° greater than those for -7 in., well rounded, Contreras Gravel.

Degree of Saturation.—There seems to be general agreement that saturated rock particles are less strong than dry ones. The degree or importance of this effect is probably inversely related to the general strength of the parent material. In 1966 Marsal (7) graphically indicated the extent of saturation effects in his Fig. 10 in which for the weakest material, designated as Gneiss Y and having a very high particle breakage during triaxial shear (see his Fig. 13), the dry frictional strength was found to be about 170 % of the saturated case. For the relatively strong basalt reported by Marsal, the dry strength was about 120 % of the saturated strength.

Despite the shortcomings previously mentioned of Fig. 1, its value appears to lie in its confirmation of a growing realization that the friction angle of a given rockfill decreases significantly as the normal pressure across the failure plane is increased. In this review, the friction angle of any triaxial test specimen is considered to be the slope of that tangent to its Mohr Circle which passes through the origin. In this definition, no recognition is accorded to such concepts as "no load shearing strength" or "apparent cohesion" of rockfill, which concepts have appeared occasionally in analyses by others of the shearing strength of coarse, granular materials.

The line in Fig. 1 which has been designated as "average rockfill" represents a judgment as to the median strength for about 100 tests on 15 varieties of rock, placed at low, medium, and high density. The use of the semi-log plot was adopted to emphasize the large variation of friction angle in the low and intermediate normal pressure ranges. If the "average" line is correctly represented on a semi-log plot by a straight line, the friction angle decreases from 55° at 1 psi to 48° at 10 psi, a change of 7° for a 9-psi increase in loading; for a further 9-psi increase, however, the friction angle changes less than 2°. Actually, the test data available suggest that the average line of Fig. 1 should not be straight but should curve upward as normal pressure decreases from 10 psi to 1 psi. This tendency has been indicated by tests on Cachuma Gravel, Infiernillo Diorite, Soledad Gravel and Infiernillo Basalt. Unfortunately, there have been no tests made at normal pressure less than 6 psi, and thus the actual relationship has only been suggested and has not yet been defined. It would be desirable if, possibly at Infiernillo, a careful investigation of rockfill strengths in the 1-psi to 10-psi normal pressure range could be undertaken with the same care and under a similar variety of placement conditions as already recorded for the intermediate and high pressure ranges.

STRESS-STRAIN AT LOW PRESSURE

There are virtually no published data on the stress-strain behavior of rockfill at normal pressures below 10 psi. Accordingly, only crude indications of strain at failure for very low pressures, as compared to that at higher pressures, can be sensed. With such limitations in mind, it can be said that the axial strain at failure for normal pressures slightly below 10 psi are apparently much smaller than those reported for tests above 100 psi, being in the order of 0.3 to 0.5 of such values. For pressures below 5 psi, the indication is than failure strains are even lower. Herein it is of course true that a very important variable at any given normal pressure would be the relative density of the given rockfill. This variable too has been investigated to a very limited degree. An example available from Marsal's work on El Infiernillo rockfill indicates a relationship which may be typical for axial strain relationships at low confining pressures (see Table 4).

Comparison with Table 4 data, dense well graded Pinzandaran Gravel failed, at a comparable lateral pressure, at an axial strain of 2.9 %, while relatively loose, poorly graded Isabella Granite failed at an axial strain of 11.5 %. In further comparison to the foregoing data, it may be particularly significant that nearly all failure strains for tests reported at lateral pressures in excess of 100 psi were in excess of 10 %, with the poorly graded materials exceeding 15 %. Thus, it is clear that failure strains for the higher confining pressures are usually two to three times those for the lateral pressure range

TABLE 4.—AXIAL STRAIN RELATIONSHIPS AT LOW CONFINING PRESSURE

Placement (1)	Void ratio (2)	Lateral pressure, in pounds per square inch (3)	Failure strain, as a percentage (4)
Loose	0.86	9.7	7.5
Dense	0.60	9.5	5.9

TABLE 5.—MODIFIED SLIDING WEDGE COMPUTATIONS OF FACTORS OF SAFETY

Case (1)	Wall width, in feet (2)	Rockfill friction angle (3)	Factor of safety (4)
Ia	4	40°	0.9
Ib	4	Fig. 1	1.1
IIa	8	40°	1.1
IIb	8	Fig. 1	1.3

below 10 psi (normal pressure range below about 17 psi). This could have particular significance in certain slope stability problems, for the larger strains which must be experienced to permit mobilizing shear strength deep within an embankment could conceivably impose greater-than-failure strains on materials near the toe of the slope, thus placing these surficial materials in a condition of incipient shear failure and possibly reducing both their density and as a result their shear strength.

APPLICATIONS IN LOW PRESSURE RANGE

One limited but particularly interesting use of a downward extrapolation of the friction angle of rockfill into the 1-psi to 10-psi pressure range of Fig. 1, is in the reevaluation of the safety of older, unconventionally designed rockfill and earth and rockfill dams in the 25-ft to 50-ft height range. Such reevaluations have been made, for example, as a consequence of Federal

Power Commission Order No. 315 which requires independent safety reviews on Federal Power Commission licensed dams.

For low and medium height, conventionally designed rockfill dams with slopes of 1.35 to 1.4:1 and with competent foundations, there is reasonable assurance as to the order of magnitude of the factor of safety of the slope, whether it be calculated by Infinite Slope, Sliding Circle or Sliding Wedge method of analysis. Depending on assumptions as to the friction angle of the rockfill, which would commonly range between 40° and 45°, the minimum computed factor of safety of a simple 1.4:1 slope would range from 1.2 to 1.4.

For unconventional designs of low rockfill dams, however, where either the upstream or downstream slope is built, for example, at 0.5:1 and retained at that slope by a laid-up, dry masonry wall of large blocks of stone with a nominal horizontal width of from 4 ft to 10 ft, it is necessary and justified to review both the stability analysis procedures and one's assumptions regarding rockfill strength. For example, the infinite slope analysis method is inapplicable. Further, any realistic sliding wedge or sliding circle must be modified for adaption to the forced direction sliding through the masonry wall dictated by the horizontal joints between blocks. Secondly, the resistance to sliding through the masonry wall is probably at least that due to friction of one cut or blasted rock surface on another rock surface, often assumed to be in the order of a 35° friction angle. Thirdly, because of the relatively low intergranular stresses involved in critical sliding wedges for dams from 25 ft to 50 ft high, a critical review of shearing strength at such low pressures is mandatory before an assignment of realistic values can be made. And fourthly, the horizontal and vertical joints in the rock masonry are normally so random in location that local outward displacement of the wall could not occur without substantial resistance to shear being mobilized by adjacent portions of the wall.

To illustrate the consequences of varying assumptions, a 40-ft high, masonry-retained, dumped rockfill with a 0.5:1 slope was examined. The masonry wall was assumed to be 4 ft wide for one case and 8 ft for a second case. The rockfill was assumed to have a 40° friction angle for both cases and a varying friction angle in agreement with "average rockfill" in Fig. 1 for both cases. The results of modified sliding wedge computations of factors of safety are given in Table 5.

The results of Table 5 are believed to be indicative of the degree of influence of the two simple parameters that were considered. The computed results are not offered as accurate determinations of the true factors of safety, which they are not, but as indicators of the degree of influence of two basic parameters, and as justification for determining these parameters with some accuracy. From the aforementioned, doubling the wall thickness is shown to increase the computed factor of safety by about 20 %. Similarly, using the average curve of Fig. 1 for rockfill strength increases the computed factor of safety by about 20 %. If it were justified to use the most optimistic strength curve of Fig. 1, this would have increased the rockfill strength another 12 %, with a further appreciable boost in the computed factor of safety.

In the foregoing calculations, the most unfavorable assumption was made concerning the effect of the masonry wall; i.e., that the laid-up blocks of rock do not interlock in any way at their horizontal and vertical interfaces. As a practical fact, interlocking to an appreciable degree exists in rock masonry by reason of the rough blasted faces and irregular shapes and thicknesses of the rock blocks. Thus, a heavy masonry wall of the type commonly used in the early California rockfill dams could not be sheared in a horizontal plane of any practical dimension without shearing through solid rock blocks. This indeterminate but very real and substantial strength in the masonry undoubtedly accounts for the fact that many dams of this design have stood solidly for over 70 yr with only the slightest trace of downstream bulging near the bases of their downstream slopes.

CONCLUSIONS

In summary, all large triaxial shear tests on rockfill specimens to date indicate that the shearing strength of rockfill, as expressed by its friction angle, varies markedly as a function of normal pressure, being high at low pressures and substantially smaller as normal pressures increase. At normal pressures less than about 10 psi, friction angles in the order of 45° to 60°, with an average of around 50°, can be expected when using good quality, clean, dumped rock. If excellent compaction is employed, friction angles in the order of 55° can be expected. Such higher-than-normally-assumed frictional strength of rockfill at low confining pressures has been found to be a substantial factor in explaining the very long time stability of low rockfill dams which were designed with very steep, dry-masonry-retained slopes.

It is hoped that the minimal amount of large triaxial test data now available in published form will be substantially supplemented in the near future, and that further testing to define relationships in the lower pressure range will be carried out and reported.

APPENDIX.—REFERENCES

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[2] Lecturer in Soil M