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JOURNAL OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION

PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

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DIVISION NAME CHANGE

The Technical Activities Committee, at its July 9-10, 1973 meeting, held in Tulsa, Oklahoma, approved the change in name of the Soil Mechanics and Foundations Division to the Geotechnical Engineering Division. However, we are continuing to use the "old" name for the Journals for the balance of 1973. The January 1974 issue will carry the new name.

CONTENTS

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DISCUSSION

Proc. Paper 10112

[a]Discussion period closed for this paper. Any other discussion received during this discussion period will be published in subsequent Journals.

TECHNICAL NOTES

Proc. Paper 10148

INFORMATION RETRIEVAL

The key words, abstract, and reference "cards" for each article in this Journal represent part of the ASCE participation in the EJC information retrieval plan. The retrieval data are placed herein so that each can be cut out, placed on a 3 × 5 card and given an accession number for the user's file. The accession number is then entered on key word cards so that the user can subsequently match key words to choose the articles he wishes. Details of this program were given in an August, 1962 article in CIVIL ENGINEERING, reprints of which are available on request to ASCE headquarters.

10170 NOVEMBER 1973 SM11

JOURNAL OF THE
SOIL MECHANICS AND
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Compaction of Hydraulically Placed Fills[a]

By Willard J. Turnbull[1] and Charles I. Mansur,[2] Fellows, ASCE

Introduction

This paper describes the personal experience of the writers and information gained from literature and other engineers, regarding the compaction of hydraulic (cohesionless) silt, sand, and sand-gravel fills. The compaction of such hydraulic fills has been given considerable consideration in recent years. The need for such consideration has been brought about by the inherent dangers of liquefaction of cohesionless sands and silts, and by recognition that in the past, sufficient attention may not have been paid to this danger to structures founded on or in hydraulic fills, particularly those structures whose failure would involve loss of life and property.

The earliest textbook reference known to the writers regarding "relative density" as a measure of the degree of compaction of a coarse-grained soil was Terzaghi's Erdbaumechanik in 1925 (17). In the early 1930's the thinking of engineers was largely centered on Casagrande's "critical density" test, even though at that time Casagrande and some other engineers, including Gilboy (4) and Taylor (16) recognized that the critical density increased with load and varied with other factors. In 1938, relative density tests were made on the shell material of Fort Peck Dam, on the Missouri River in Montana. In 1937, the relative density test was used along with Casagrande's critical density test on the Franklin Falls Dam, although at that time the former was called the "degree of compaction" test. The relative density test was described by Wilhelm Loos (8) in a paper in 1936. However, it was not until the late 1950's and

Note.—Discussion open until April 1, 1974. To extend the closing date one month, a written request must be filed with the Editor of Technical Publications, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 99, No. SM11, November, 1973. Manuscript was submitted for review for possible publication on March 14, 1973.

[a]Presented at the October 16-20, 1972, ASCE Annual and Environmental Engineering Meeting, of the Soil Mechanics and Foundation Division, held at Houston, Tex. (Preprint 1802).

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the early 1960's that the relative density test came into fairly common use. One of the greatest drawbacks today in the use of relative density is that the test has not been standardized. All too often different laboratories use different procedures that do not yield consistent test results.

Terzaghi and Peck in their 1948 textbook (18) correlate the standard split-spoon resistance to the degree of density of coarse-grained soils; an adjective rating only, was developed. Gibbs and Holtz, in a paper (3) in 1957, developed a mathematical relationship between the standard penetration resistance and relative density, which included an empirical correction or allowance for load. Peck and Bazaraa (11), in an examination in 1969 of a paper by D'Appolonia, et al. (2) in 1968, developed a mathematical relationship between standard penetration resistance and relative density in which load was considered. At the present time, most engineers concerned with liquefaction of coarse-grained soils think in terms of percentage of relative density. In specifications, the requirement for relative density is usually set somewhere between 50% and 80%. The lower value is generally considered adequate where bearing capacity, or settlement, or both, is not particularly critical; the higher value may be required when major structures are to be supported on the fill and little or no settlement can be tolerated and when the fill may be subjected to high intensity earthquakes or shock.

As previously stated, the writers have drawn on their own experience and that of others on projects with which they were acquainted during construction. It also happens, unfortunately, that actual post-construction reports were not written on most of the projects, and construction data have either long since been destroyed or misplaced. In consequence of this, it is necessary to resort to memory.

In the following paragraphs, several projects are analyzed. Note that the term "hydraulic fill" has been broadened to include projects that: (1) Might be described as puddled fill; and (2) might be described as bulldozed fill in water.

Hydraulic Fills

Fort Peck Dam.—The data summarized herein were taken from a report dated July, 1939, concerning the slide of a portion of the upstream face of Fort Peck Dam and on knowledge gained by the senior writer during construction of the dam from visual inspection and conversations with Middlebrooks. The shell material was a fine to coarse sand containing very little gravel. The shell material had an average effective size of 0.17 mm, a uniformity coefficient of 1.9, and 5% passing the No. 200-mesh sieve (see Fig. 1).

The dam was constructed by the hydraulic method with four 28-in. electrically operated pump dredges. The pumped material was discharged principally through trap pipe lines along the outside edges of the shells, thus forming beaches sloping toward a central core pool, with the coarsest material being deposited on the outside of the shell and gradually grading finer toward the core pool. In the lower part of the dam some double or "hairpin" discharge lines were used on each edge of the dam (see Fig. 2). In the lower and closure sections of the dam, the "table method" end discharge procedure was used to some extent. Near the pool the beach slopes were as flat as 2%, but the slopes

were steeper toward the outer edges of the embankment. Shear or batter boards (see Fig. 3) were used extensively for directing and breaking up concentrated flows toward the core pool.

A compacted test section of hydraulic fill was constructed to investigate the feasibility of (track) tractor compaction with a 95-hp tractor operating at full and at half speed. The test section was 6 ft to 8 ft (1.8 m to 2.4 m) deep and was constructed in 6-in., 12-in., and 18-in. (152-cm, 304-cm, and 462-cm) thick layers with each layer being compacted by three, six, and nine passes

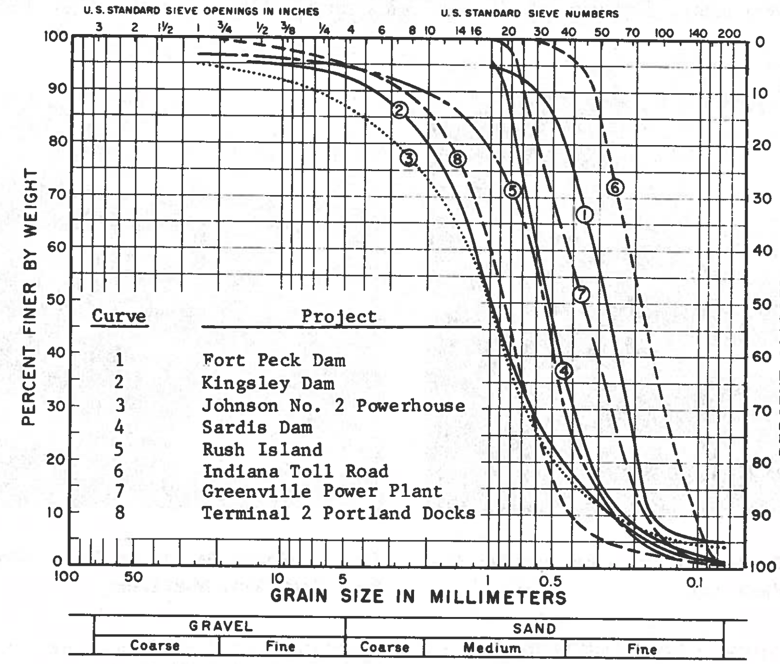


FIG. 1.—Average Gradation of Hydraulic Fill Materials

of the tractor. The test results indicated the following:

1. Except for the thicker layers, most of the compaction was obtained in three passes.
2. Generally, the maximum density was obtained with the tractor running at full speed.
3. The maximum density was obtained in the 6-in. (150-mm) layer.
4. With three passes, the density ranged from 101.3 pcf to 95.8 pcf (1,625 kg/m^3 to 1,535 kg/m^3) respectively, for the 6-in. (150-mm) and 24-in. (610-mm) layers. The corresponding relative density was 92% to 67%. In general, the same trend held with an increasing number of passes but narrowed, with density increasing up to nine passes principally for the thicker layers.

The maximum density for the relative density test was determined in the laboratory by placing the sample in the container on a pedestal and jarring it up and down by a cam on a motor-driven shaft. The minimum density was obtained by gently pouring the sand through a funnel into a graduated glass cylinder. Both tests were conducted in the dry and saturated conditions. The dry maximum density proved to be the greatest and any further analysis applies to it.

The average relative density of the shell as determined in two tests pits was 65%, with the average density being 94.4 pcf (1,510 kg/m^3).

Critical density tests were conducted at Harvard University, Cambridge, Mass., the Waterways Experiment Station, Vicksburg, Miss., and the Fort Peck

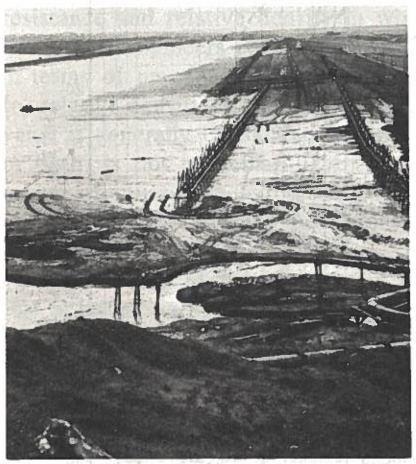


FIG. 2.—Hairpin Trap Lines and Beach—Fort Peck Dam

FIG. 3.—Shear Boards on Beach—Fort Peck Dam (Note Mud Balls)

laboratory at Fort Peck Dam. These tests were conducted at vertical pressures ranging from about 2 tons per sq ft to 45 tons per sq ft (192 kN/m^2 to 4,320 kN/m^2). The density tests in the fill to which the laboratory tests were compared were under loads ranging from about 5 tons per sq ft to 11 tons per sq ft (480 kN/m^2 to 1,050 kN/m^2). In the latter range of loading, the laboratory critical densities ranged from about 97 pcf to 91 pcf (1,550 kg/m^3 to 1,460 kg/m^3) with an average of 94 pcf (1,510 kg/m^3).

As the (sand) shell material was being deposited at densities slightly greater than the critical density and since the use of rollers would hamper fill operations, the decision was made not to use rollers. However, after failure of a portion of the embankment, the fill was compacted.

Kingsley Dam.—Part of the data from Kingsley Dam, Central Nebraska Public Power and Irrigation District, are taken from the revised design analysis report (19). The average gradation of the shell material appears in a textbook authored by Justin, Hinds, and Creager (7).

Kingsley Dam was constructed by the hydraulic fill method with two 36-in. (915-mm) electrically operated dredges, one upstream and one downstream. The dredged material was discharged from trap pipelines located on trestles near

the outer edge of each shell as in Fig. 4. The shell borrow pits did not contain sufficient fines to build the core; consequently, it was necessary to obtain additional fines from an independent source to make up the deficiency. This

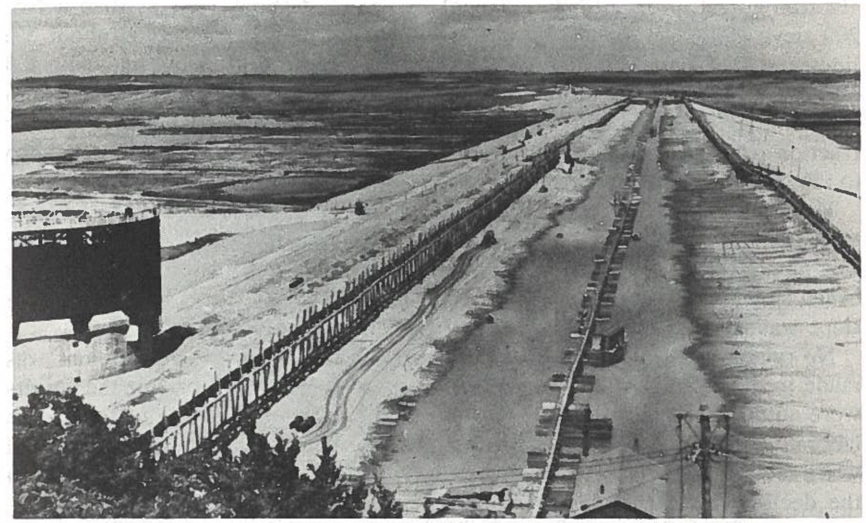


FIG. 4.—Traplines on Beach Edges and Floating Trap Line in Core Pool—Kingsley Dam

FIG. 5.—Shear Boards on Beach—Kingsley Dam

latter material was hauled and dumped into a sluice box on top of the south abutment and was discharged into the core from a floating trap line along the core pool center line. Shear or batter boards, as in Fig. 5, were used to break up heavy flows into the core pool. A small (track) tractor was also used to assist in this operation.

The average uniformity coefficient for the shell material was about 5.0 and the effective size about 0.25 mm. The average shell material contained about 7% fine gravel up to 1/2 in. (13 mm) in size, and about 1% less than a No. 200 sieve (Fig. 1). The shell material in the borrow pits contained about 5% fines and 10% fine to coarse gravel. The uniformity coefficient was about 6.5 and the effective size about 0.14 mm. When this material was pumped onto the dam, most of the fines were washed into the core pool and the coarser fraction of the coarse material, as it built up under the pipeline, was pushed to the outer edges by the bulldozer to form a filter blanket for the riprap.

The dry unit weight of the shell material averaged about 111 pcf (1,780 kg/m^2) and that of the borrow material about 120 pcf (1,920 kg/m^3). The difference was obviously due to the better gradation of the borrow material in situ. The beach slope was as steep as 10%, particularly near the pipeline, but the average slope was about 5%.

No relative density tests were made of the shell material. An attempt was made to determine the critical density using a direct shear box but the results were quite erratic, undoubtedly because of the coarseness of the material. The laboratory tests, however, did indicate that the density of the fill as deposited was a few pounds greater than the critical density. On the basis of this information the decision was made (inclusive of the Board of Consultants consisting of Hunt, Justin, and Creager) not to compact the material as hydraulically deposited. This decision was also influenced by the fact that good distribution of material could be maintained using the trap discharge technique, thereby ensuring uniform thin layers of deposition which would tend to eliminate air entrapment. Also, as the material was free draining, there would be a considerable downward seepage force. In addition, all beach material was deposited above water; in this case, the core pool. Deposition of the material above water ensures a denser and cleaner shell.

Two flow slides occurred in this fill: one during construction and one after completion while filling the reservoir, both of which demonstrated the wisdom of properly depositing, or compacting hydraulic fills, or both. A brief description of these flow slides is given subsequently.

In constructing the stilling basin, a considerable amount of over-excavation had occurred. The river side of the excavated basin was in alluvium and the contractor drove sheet piling to retain the alluvial sand-gravel. The backfill between the river-side (stilling basin) wing wall and the sheet piling consisted of sand-gravel dumped on the edge of the area to be filled and simply pushed into the excavated area from the top until the bulldozer could be worked out on the fill at ground level. The sheet piling was pulled as the fill progressed. The riprapped bank downstream of the stilling basin was also built in this manner. Undoubtedly, the dumped sand-gravel fill not only bulked but entrapped much air. It is estimated that the fill had a relative density not over 25%. The contractor assembled his dredge in the wintertime about 600 ft (180 m) downstream of the stilling basin but could not launch it because of a thick layer of ice in the channel. He wanted to break the ice by blasting but the resident engineer limited the size of dynamite charges because of fear of cracking the walls of the stilling basin. The contractor on his own iniative greatly increased the charges

FIG. 6.—Liqufaction Failure on South Side of Morning Glory Spillway—Kingsley Dam

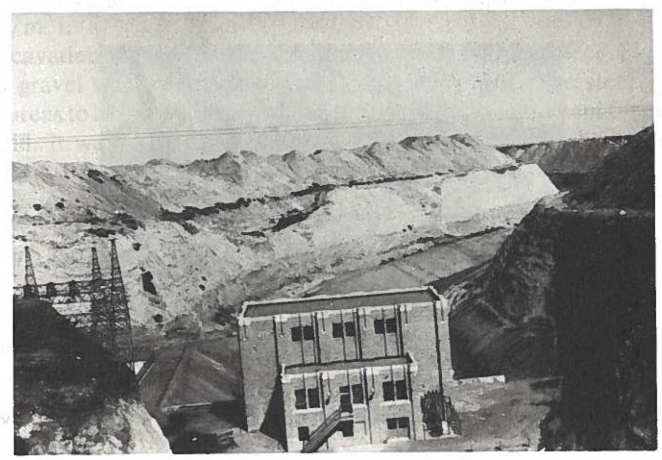


FIG. 7.—Johnson No. 2 Powerhouse with Backfill in Place (Note Steep Slopes)

and managed to break up the ice so he could launch the dredge. The concrete walls of the structure were not damaged but the bulldozed sand-gravel fill liquefied and the riprap on the bank moved out with the liquefied material into the channel. The material behind the wing wall flowed out following the bank material. A noticeable feature was that the in situ sand-gravel did not liquefy but remained

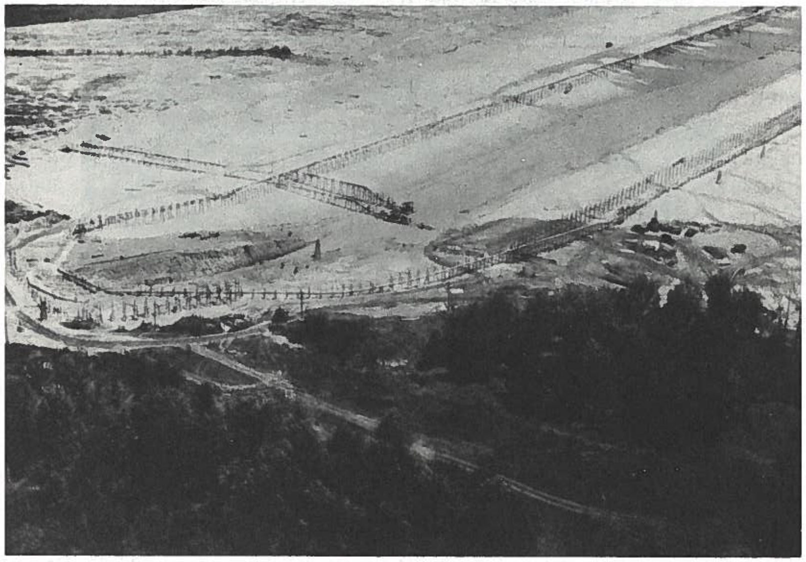


FIG. 8.—Traplines on Beaches and Around End of Core Pool—Sardis Dam

FIG. 9.—Vibrating Roller for Compacting Hydraulic Fill for Rush Island Power Plant

standing practically vertically.

The other flow slide occurred in February of 1942, during the usual warm period which occurs during this month. The morning glory spillway, about 100 ft (31 m) in diameter, is located about midway of the upstream slope. During construction, the dredge line was moved from the low side of the spillway to the high side, leaving a triangular prism on each side of the structure which

could not be filled hydraulically. These two prisms, about 25 ft (7.6 m) deep at the structure tapering to zero 200 ft (61 m) from the structure, were filled by bulldozing sand-gravel from the slope above. The material was not sluiced or compacted. Again the material undoubtedly bulked since its moisture content was only 5%, and large quantities of air were undoubtedly entrapped. A drill rig was used to install a piezometer through the prism of sand on the south side of the structure; while installing the piezometer, the rear end of the drill rig sank about 30 in. (760 mm), indicating the looseness of the material. When the flow side occurred, the water level of the reservoir was above the bulldozed soil prisms and the ice cap was about 3 ft (0.9 m) thick. During the night of the flow slide, cracks in the ice began to develop, probably due to the early thaw, or the rising water level, or both. When a crack developed there was a loud report which could be heard 10 miles (16 km) distant. Two of the cracks extended into or closely adjacent to the spillway structure. It is believed that the shock waves produced by the ice cracking caused the loose sand-gravel in the two prisms to suddenly consolidate and thus liquefy and flow down the dam face to the toe of the slope. Soundings indicated that the material which flowed out closely approximated the area of the two prisms (see Fig. 6).

Johnson No. 2 Powerhouse Backfill.—Johnson No. 2 Powerhouse is located on the lower end of the power canal of the Central Nebraska Public Power and Irrigation District. The powerhouse is founded on coarse sand and gravel below the water level and except for one clay layer about 3 ft (0.9 m) thick and a surface layer of loess about 6 ft (1.8 m) thick, the entire excavation for the structure is in coarse sand grading into sand-gravel with depth. The average sand-gravel material was well-graded and had less than 5% fines, an effective size of about 0.25 mm, uniformity coefficient of about 5.5, and about 15% of gravel (Fig. 1). The standard Proctor density averaged about 125 pcf (2,000 kg/m^3).

The excavation slopes for the structure were quite steep (see Fig. 7) as the sand and gravel was well-packed and slightly cemented. The steep slopes and confined areas to be backfilled restricted the use of large equipment for compacting the backfill. It was decided to place a filter around the powerhouse and puddle the sand-gravel backfill. The foundation and excavated slopes were pervious. The excavation was dewatered and the filter brought up around the walls of the powerhouse just ahead of the puddled fill.

The method of puddling was as follows:

1. Loose sand-gravel was slid down the slopes from stockpiled material with shovels and caught in wheelbarrows.
2. As the wheelbarrows were dumped, the sand-gravel was raked back and forth in about 6 in. (150 mm) of water maintained on top of the gravel.
3. There was no tamping of sand-gravel.

The surface of the material became so dense that the tire of the loaded wheelbarrow did not sink into the sand-gravel. Densities up to 140 pcf (2,240 kg/m^3) were measured, which is estimated to represent over 85% to 95% relative density.

Sardis Dam.—Sardis Dam in Mississippi was constructed primarily as a hydraulic fill by the Corps of Engineers with hired labor. Part of the data given in this paper were taken from a report (13) with limited distribution prepared by W. B. Nelson, Operations Engineer on Sardis Dam. The remaining data, particularly as to fill densities, were furnished by R. C. Baker, Chief of Engineering on the dam. The fill was constructed with one 24-in. (610-mm) dredge with two 24-in. (61-mm) pumps connected in tandem. The fill was constructed with trap lines close to the outer edge of the core pool (see Fig. 8). Shear or batter boards were used on the fill to control and direct the flow of water to the core pool.

As the fill was placed it became apparent that there was not enough core material in the borrow pit. This deficiency in core material was made up by hauling core material from an outside source and dumping it where needed in the borrow pit. The sand graded from fine to coarse and as deposited in the shell had an average uniformity coefficient of about 2.7 and an effective size of 0.22 mm. The amount of fines in the shells averaged about 2%. Very little of the material was coarser than coarse sand (Fig. 1).

A compaction test using (track) tractors was made soon after starting placement of the hydraulic fill. This test showed that the density of the fill could be increased with tractor compaction. However, it was decided not to compact with tractors since the densities being obtained by the hydraulic fill procedure itself were slightly greater than the critical density of the sand.

Rush Island Steam Generating Plant, Union Electric Company, St. Louis Mo.—A (sand) fill for the general plant area, approx 5,000,000 cu yd (3,800,000 m^3) in volume and 22 ft (6.7 m) high, was required for the Rush Island Steam Generating Plant to protect it against flood waters of the Mississippi River as no protective levee exists between the plant site and the river. The portion of the fill for the powerblock, completed in 1971, was built within retention levees constructed of compacted clayey soils. The powerblock fill, containing about 650,000 cu yd (497,000 m^3) of sand was dredged directly from the Mississippi River. Two dredges, one 18 in. (460 mm) and the other 15 in. (380 mm) were used to pump this fill. The end discharge method was used. The sand fill contained less than 1% fines with the gravel content ranging from zero to 25%, averaging about 6%. The average 60% size was about 0.62 mm and the average 10% size, 0.28 mm; the uniformity coefficient was approximately 2.2. The average gradation of the Rush Island powerblock fill is shown by the curve plotted in Fig. 1. (The preceding data were taken from Ref. 9.)

The specifications for the powerblock fill required an average relative density of 65% with not more than 10% of the record samples having a relative density less than 60%. Initial testing of the fill indicated that an average relative density of 65% could not be obtained without some compaction during dredging. It was found that the specified density could be obtained by tracking 12-in. to 15-in. (300-mm to 380-mm) lifts with five coverages of a D-6 or D-8 (Caterpillar) tractor, or three passes of a vibratory steel drum roller. Three passes of a rubber tired payloader in conjunction with two passes of the vibrating roller produced relative densities ranging from 65% to 70%. One or two passes of the vibrating roller on a 12-in. to 15-in. (300-mm to 380-mm) layer was found to give in most cases the specified 65% relative density. Densities taken in test pits showed that the vibrating roller kept increasing the density to a depth of about 3 ft (0.9 m). The vibrating roller (Fig. 9) was used on the majority

of the fill assisted to some extent by a Caterpillar tractor which was maintained on the fill to direct the flow of dredge water and to prevent ponding.

The laboratory maximum densities were determined for the most part by compacting a sample of fill material taken immediately adjacent to the record density sample according to ASTM (Modified) D1557-70 except that 75 blows of the hammer were applied to each layer instead of the 55 blows called for by ASTM D1557-70. Check tests made for maximum density using the ASTM D-2049 (vibrating table) procedure gave maximum densities of −1.1 pcf to 3.9 pcf (−18 kg/m^3 to 63 kg/m^3) more than achieved by the compaction method used. Thus, the relative density of the in situ fill was probably about three percentage points less than values that would have been obtained based on maximum densities using ASTM D-2049. The minimum densities were obtained by carefully pouring a sample of the (dry) fill sand into a 6-in. (150-mm) mold through a funnel and a 1-in. (25-mm) hose in accordance with ASTM D-2049 procedures. Since the fill material averaged about 6% gravel sizes and ranged

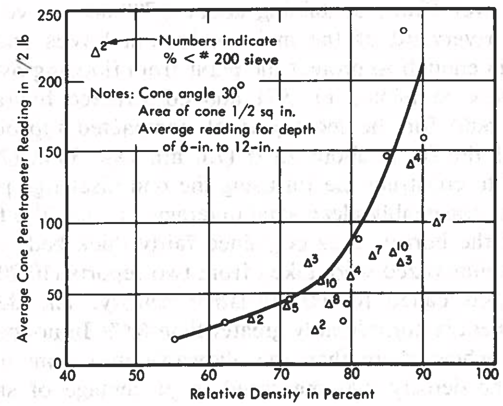


FIG. 10.—Correlation Between Airfield Cone Penetrometer and Relative Density; Greenville Fill—Mississippi Power and Light Power Plant

up to 25%, there was considerable stratification of the fill, which made representative undisturbed sampling difficult. Unfortunately, in a stratified fill it is difficult to correlate the field density with the relative dnesity because the in situ fill exists in a stratified condition while the control testing results in the coarse and fine sand becoming mixed together. Thus, the actual relative density of the in situ fill would be some greater than the "apparent" relative density computed from the maximum and minimum laboratory tests on the mixed sample. Therefore, in taking control samples from the fill, an effort was made to take samples where there was a minimum of such stratification, assuming that if unstratified sands had adequate density the stratified sands would also have adequate density.

In general an excellent fill was obtained with an average density equal to about the 65% relative density specified.

Contract C-6 Indiana Toll Road.—A portion of the Indiana Toll Road (about 1,000,000 cu yd) (700,000 m^3) was constructed by hydraulic fill methods using an 18-in. (460-mm) suction dredge and end discharge pipe. The data were taken from two reports [(6,15), reports furnished through the courtesy of S. J. Johnson, Soils and Pavement Laboratory, U.S. Army Waterways Experiment Station, Corps of Engineers, Vicksburg, Miss.]. The gradation of the fill material is not available but the material was a very uniformly graded sand.

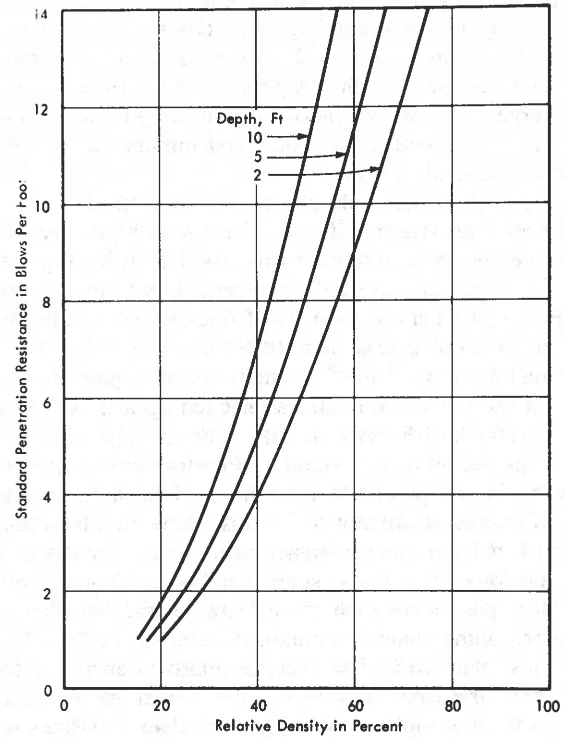
The uniformity coefficient of the borrow material averaged slightly less than 2.0 ngl, the effective size somewhat greater than 0.1 mm. The fill material contained very little gravel and only about 1% of fines (Fig. 1). The normal hydraulic fill operations resulted in low densities which did not meet specification requirements. In order to achieve the required density, two D-8 bulldozers were used to systematically track the fill. This resulted in achieving densities of 98% or more of those shown by the Michigan cone test (5). The corresponding relative density is not known but experience has shown that a 100% Michigan cone test density is greater than that given by the modified Proctor compaction test.

Greenville Power Plant, Mississippi Power and Light Company.—The fill for the Greenville Power Plant, containing about 2,700,000 cu yd (2,100,000 m^3) was constructed riverward of the main protection levees and consequently, required a fill high enough to protect the plant from flooding by the Mississippi River. The fill was completed in 1971 and constructed hydraulically behind retention levees built for the most part of compacted topsoil material. The average height of the fill is about 25 ft (7.6 m). One 30-in (760-mm) suction dredge was used to construct the fill using the end discharge procedure. Most of the borrow was reasonably clean sand (averaging about 20% fines); however, other portions of the borrow area contained fairly thick beds of silt and clay. The data herein summarized were taken from two reports (10,20).

The specifications called for 60% relative density. The densities actually obtained were generally considerably greater than 60%. In no-load or noncritical areas of the fill where more than the allowable maximum of 10% of fines was permitted, the density was measured in percentage of standard Proctor maximum density

The field densities of the hydraulic fill were measured with 4-in. (100-mm) drive tube samples taken 6 in. to 12 in. (150 mm to 300 mm) below the surface of the fill. An airfield cone penetrometer was used to some extent after being correlated with relative density (see Fig. 10). A split-spoon sampler was also used a limited amount in evaluating the fill density in some areas (see Fig. 11).

The fill was rolled with two very heavy rubber-tired rollers (Caterpillar log handlers, Fig. 12) with large drive wheels, and one D-6C Caterpillar tractor, all with bulldozer blades. The tractor bulldozer was also used to fill gullies and to throw up small levees for directing the flow of water. It was also used in protecting the drains against erosion. These rollers were operated continuously in a systematic manner to obtain uniform coverage of the fill. The hydraulic fill slopes varied from 1 on 20 to 1 on 50. Fill drainage was from two sluice boxes equipped with stoplogs on each side of the fill near the end opposite from the starting point. The tractors were very effective in squeezing out layers of trapped silt if they were not over 4 ft (1.2 m) deep. They were also very effective in compacting the hydraulic fill whether it contained more than 10% of fines or not.



Note: Relative density values are computed for a saturated sand having a wet unit weight of 122.5 pcf. (Ref. - Peck, R.B. and Bazaraa, A.R.S., discussion in Journal of Soil Mechanics and Foundations, ASCE, May, 1969)

FIG. 11.—Relative Density Versus Standard Penetration Resistance

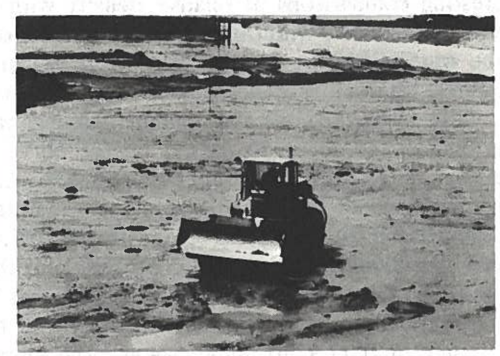


FIG. 12.—Compactive Hydraulic Fill with Rubber-Tired Rollers—Greenville Fill—Mississippi Power and Light Power Plant

Relative densities were determined in the laboratory as follows: The minimum density was obtained by gently pouring dry sand into a 1.25-in. (32-mm) diam by 3.5-in. (89-mm) long cylinder topped by a 3-in. (76-mm) long surcharge cylinder. The maximum density was determined on dry sand in the same cylinder by alternately placing and vibrating three layers, with the third layer filling to the top of the surcharge cylinder. Vibration was accomplished by contacting the exterior wall of the mold with a thin steel rod connected to and vibrated by a hand-operated mechanical "engraver."

Sieve analyses made on the hydraulic fill showed an effective size of 0.18 mm and a uniformity coefficient of 2.4. There was relatively little gravel in the fill; where gravel was encountered it was less than 10% in quantity. Relative density tests were made on samples containing less than 10% fines. (About 70% of the samples contained less than 5% of fines.) The overall average relative density of the fill containing less than 10% fines was 75%. The fines in the no-load or light load portions of the fill sometimes ran higher than 10% (average of about 20%) and the fill was sometimes checked against both relative density and percentage of standard Proctor density. The average relative density was 65% and the average percentage of standard Proctor density was 96%.

The top stratum in the powerblock or heavy load area was excavated to sand. After an unsuccessful attempt to fill this excavation hydraulically, it was then decided to fill it by normal compacting methods. Sand was hauled from a pit relatively free of excess fines, spread in 12-in. (300-mm) loose lifts, and compacted by three passes on each lift by (track) tractors after saturation of the lift with water. Some samples contained as much as 20% fines but most of them showed less than 10%. The average relative density of those samples containing less than 10% fines was 79% and the average percentage standard Proctor density of those samples containing more than 10% fines was 96%.

The average relative density determined by the cone penetrometer (using the curve shown on Fig. 10) was checked in the field by three tests closely spaced around the drive tube sample against a group of soils which had the relative density made in the normal manner. The former showed an average of 74% while the latter showed an average of 78% relative density.

Some other interesting comparisons of relative density with the split-spoon and cone can be made. A test made with the split-spoon sampler on uncompacted fill placed below the water table showed the fill to have an estimated relative density of 10% to 30%. The senior writer made many tests on uncompacted fill above the water table using the cone and obtained readings ranging between 10 and 30 with an average of about 20. Based on the curve shown in Fig. 10, the relative densities ranged between about 50% and 60%. A separate direct density test on an uncompacted fill placed above the water table outside of the main fill area showed an average relative density of 60%. The gradation of this latter material was identical to that shown on Fig. 1 for the main fill.

Terminal No. 2, Portland Public Docks.—The fill for Terminal No. 2, Portland Docks, Oregon, was basically a hydraulic fill with up to 40 ft (12 m) being placed under water and 26 ft (7.9 m) above water. The purpose of the fill was to serve as a foundation for a dock and bulkhead about 1,400 ft (430 m) long. The material was pumped from the river close by. A stockpile of the material was pumped for preloading certain areas of the fill. Some of this stockpiled material was subsequently bulldozed over the edge of the fill to bring the slip side of the fill, its north corner, and a short section of the fill southward, out to the design grade line. The veneer of bulldozed fill was 18 ft (5.5 m) thick in some places.

Material for this paper was obtained by the senior writer who acted as a consultant to Cornell, Howland, Hayes, and Merryfield during the latter portion of the fill work when the slope stability came under question from a report by Cornforth of Shannon and Wilson (1), who were consultants on design of the dock, and a report by Schroeder and Byington (14).

The dredged fill contained about 5% gravel and very little fines (Fig. 1). The effective size of the material is about 0.4 mm and the uniformity coefficient about 2.5.

While attempting to excavate the underwater fill toe to grade, several slope failures occurred. Split-spoon penetration resistances indicated that these slopes had penetration resistances at various elevations as low as 2; however, Schroeder and Byington show an average of 5, which when transformed to relative density by the procedure developed by Gibbs and Holtz (3) represented an average relative density of about 38%. Because of the instability of the slopes, it was decided to densify the slopes by driving Class C timber piles. Over 1,000 piles were driven on a spacing generally ranging from 4 ft to 5 ft (1.2 m to 1.5 m), but with some spaced up to 10 ft (3 m). The resulting densification showed an average penetration resistance of about 10 or an average relative density of about 55%. The hydraulicked sand above the water table which was compacted by directed equipment travel had an average penetration resistance of about 30, which represents a relative density of about 90%. The dock has been completed and put in service without any further trouble.

Miscellaneous Compaction Methods.—Numerous other methods of compaction of loose sands under water exist but with which the writers have had only minor personal experience. Some of these are: vibroflotation, vibrating probe, explosives, and underwater equipment similar to above-water equipment. Johnson, et al. (6) have summarized the experiences recorded in the literature with the preceding procedures and this will not be repeated herein. One graph in the paper by Schroeder, et al., shows their summarization of the literature on vibroflotation, compaction piles, and the vibrating probe. This graph shows the upper limit of compaction of sand by the vibrating probe and compaction piles to approximate the lower limit of compaction by vibroflotation, with compaction piles tending to be more efficient than the vibrating probe.

Summary and Conclusions

This summary and conclusions apply principally to hydraulic sand fills placed above water for which compaction may or may not have been specified, depending upon the relative density required. It appears that in a well-controlled hydraulic fill with less than 10% fines, a relative density of 50% to 60% can be obtained with no compaction. Some of the requirements for a well-controlled hydraulic fill are:

1. The true water table must be well below the surface of the fill so that there is downward drainage at all times.

2. Material must not be bulldozed into low places without subsequent sluicing; otherwise low densities will result.

3. A uniform flow must be maintained over the fill surface by use of bulldozers and shear boards to direct the flow of water. Trap discharge lines are definitely suqerior to end discharge lines in this respect.

4. Deposition into pools of water must be prevented in order to eliminate the accumulation of fines and low densities.

If hydraulic fills require relative densities greater than 50% to 60%, then compaction rolling above the water table is required. This is not difficult and may be accomplished with (track) tractor or rubber-tired rolling, or with a steel drum vibrating roller, to name just a few of the more common ones.

A hydraulic puddled fill can be successfully obtained if the material being puddled is relatively free-draining and the true water table is well below the perched puddling water table so that a vertical component of drainage is maintained. One additional requirement is that the material as deposited in the water be manipulated by some means to allow all air to escape to the water surface.

All hydraulic fill placed under water, other than that described in the preceding paragraph, will require some form of compaction to bring it to a stable density. Some of the more common procedures for such compaction after the fill is completed are: (1) Vibroflotation; (2) compaction piles; and (3) vibrating probes. A less common procedure is underwater (compaction) rolling as the fill is placed.

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Three-Dimensional Finite Element Analyses of Dams[a]

Closure by Guy Lefebvre,[6] James M. Duncan,[7] M. ASCE, and Edward L. Wilson,[8] M. ASCE

The writers thank Krishnayya and Eisenstein for their discussion, pointing out that the conclusions regarding the usefulness of plane-strain analyses of the longitudinal section apply only to homogeneous embankments. When the dam is not homogeneous, neither a longitudinal section through the shell nor a longitudinal section through the core gives results representative of the entire embankment, because the movements and stresses in the dam are determined by the properties of both the core and the shell. The movements within the core calculated by a three-dimensional analysis will be intermediate between those calculated by a plane-strain analysis performed using the properties of the core and a plane-strain analysis performed using the properties of the shell throughout the section. The stresses within the core calculated by a three-dimensional analysis will not correspond to those calculated by a plane-strain analysis of the longitudinal section, no matter what properties are used in the plane-strain analysis, because the important effect of upstream-downstream arching is completely absent from the plane strain analyses.

The analyses described in the paper were performed using the CDC 6400 computer at the University of California, Berkeley, Calif. The total amount of computer time used for all of the analyses was approx 2 hr. Note that because of symmetry, it was only necessary to analyze one-quarter of each of the dams described in the paper. The number of degrees-of-freedom, the band width, and therefore the computer time were considerably less than would have been required to analyze the full dam in each case.

[a]July, 1973, by Guy Lefebvre, James M. Duncan, and Edward L. Wilson (Proc. Paper 9857).

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Compaction of Hydraulically Placed Fills[a]

Discussion by Henry M. Reitz,[3] F. ASCE and Alfred H. Hunter, Jr.,[4] M. ASCE

This paper presents quantative data from actual large construction projects and is, in part, a state-of-the-art type summary for which we compliment the authors. The potential problems of fills placed hydraulically and the factors during construction which may influence them are well stated. The lack of adequate factual data on the quality of completed fills placed by hydraulic methods is most unfortunate.

During 1970, two fills of approx 333,000 yd each were made for supporting buildings on shallow foundations on sites along the Missouri River. During 1972, a plant fill of approx 3,500,000 yd was placed for storage yards, railroad trackage, and ancillary facilities to the Rush Island electric generating plant.

The writers had the opportunity to develop considerable data during hydraulic fill placed for the plant fill on the Rush Island project and were familiar with the initial fill for the power block only, which the authors included. This discussion concerns only hydraulic fills placed in a flood plain to raise a site for material storage or one-story manufacturing or warehousing. The fills were dredged from adjoining areas in the flood plain and pumped to the points of deposition with especial attention given to the discharge locations and control of maximum pool water levels at the toe of the fill actively being placed.

The authors' summary and conclusions limit the characteristics of a well-controlled hydraulic fill without any specific compaction equipment as a maximum relative density of 50%-60%. The writers' experience has indicated that well-controlled hydraulic fills, without separate mechanical compaction can have median relative densities above 70% with only a very small percentage of densities below 50%. In addition, experience shows that limitation on fines in the borrow material is redundant for this type fill even where dredging includes a clayey silt overburden.

The Rush Island contract for the plant fill, in several aspects, appeared to be somewhat unusual for American practice. The parameters for the fill and limits on the operation were discussed and mutually agreed to by the owner, his prime consultant and contract manager, the prime contractor for the site development work, and the dredging subcontractor. The fill was dredged from an adjoining area which volume of borrow pit has the potential of being used as an ash disposal pond. Since the fill was used for transportation routes, light buildings, and storage, relative density was the only important physical characteristic. During the construction, verification of quality of the material in place was checked continually by at least two distinctly different interests, 1 and 2 as one and 3 and 4 as the other: (1) A nationwide physical testing firm was employed for the owner by the prime consultant; (2) the prime consultant had

[a]November, 1973, by Willard J. Turnbull and Charles I. Mansur (Proc. Paper 10170).

[3]Pres. Reitz & Jens, Inc., St. Louis, Mo.
[4]Vice Pres., Fruco & Assoc., Inc., St. Louis, Mo.

at least one experienced person continually assigned to this phase of the plant fill-contract; (3) the prime contractor had an in-house soil mechanics-foundation group—at least one of its experienced staff was assigned in the field; and (4) the dredging contractor who was a subordinate to the prime contractor, hired the senior writer's firm for technical guidance and continual field inspection and testing to assure contract compliance.

During the prior contract for the power block fill, using a river-bottom source, field testing did not show acceptably uniform testing results for apparent constant characteristics of fill and ways of handling the material after it was discharged on the fill area. It was not brought out by the authors that pumping was required to maintain drainage in the confined power block area. Efforts, during the initial portion of the plant fill construction, to find parameters that could reliably indicate the maximum and minimum densities for the relative density computation, were frustrating (21).

The specific nature of the numerical reporting of relative density when compared with the determination of "percent compaction" show that it is much more sensitive to minor variations in field dry density. This sensitivity indicates a unique and more troublesome characteristic in controlling construction operations where specifications are based upon relative density. As a result, the numerical data reported for this fill should not be based on correlations from the Standard Penetration Testing "*N*" values but rather on actual field densities compared with an individual maximum-minimum determination for each field density test. A further advantage of quantative evaluation of the fill during its construction is the visual observation of the filling process. The role of visual observation by experienced, qualified field personnel is invaluable in the writers' opinion, an opinion also expressed by the senior author. When observation is used to supplement physical testing objectively, the results should be a good description of the actual quality of the fill in quantative terms.

The writers have, at least, 125 field tests plus a like number of maximum and minimum densities and gradational characteristics of the samples removed for the field tests. The relative densities, expressed in decile increments for these 125 tests, show two tests with relative densities less than 40%; two tests between 40% and 50%; six tests between 50% and 60%; 11 tests between 60% and 70%; 18 tests between 70% and 80%; and 86 tests above 80% relative density.

These data on the Rush Island plant fill, as tested during close surveillance of the field operation, suggest that the quality of fill that can be placed, using good construction and engineering practice, is better than has been indicated in the literature or by the authors heretofore. Secondly, it has sufficiently higher relative density so the specifications placing unnecessarily tight requirements on the unacceptable fineness or requirements for densification of the fill by some mechanical means cannot be justified if used indiscriminately. This satisfactory experience of constant density fills for the purposes enumerated has been shown for central United States. There are other areas in which the nature of borrows and the care by contractors even for relatively small volume of fills have been sufficient to give a good constant volume end result without mechanical densification.

In the writers experience, the determination of relative densities for fills of cohesionless material has been by the use of the Gibbs and Holtz criteria,

based upon Standard Penetration numbers, either with or without modification. The writers strongly urged that where fill quality can be tested as the fill is being placed, the more reliable determinations of relative densities by physical testing should be used. As is the case in almost all construction projects, the additional cost for the technical services necessary to determine the characteristics of the fill as it is being placed as compared to attempting to establish the quality by correlation after the fill has been completed is small.

For the Rush Island plant fill, a comparison of the contract costs, if awarded as originally bid (utilizing river borrow) but rejected by the owner, with the finally negotiated contract, was in the ratio of approximately 2 to 1. Since this plant yard cost is a capital charge that enters into the rate base for the utility, it represents a saving to society of even greater magnitude.

Appendix.—Reference

21. Reitz, H. M., "Correlation Between Gradational Parameters and Limiting Densities for Cohesionless Materials Placed Hydraulically," *STP523, American Society for Testing and Materials*, July, 1973, pp. 444–454.

Strain-Stress Relations and Failure of Anisotropic Clays[a]

Errata

The following correction should be made to the original paper:

Page 1102, Table 2, Col. 4: The last two numbers should be interchanged

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Tieback Wall Construction—Results and Controls[b]

Discussion by Robert E. White,[4] F. ASCE

The authors have made a valuable contribution to the design of tiebacks in earth, which is still more or less based on empirical methods. Actual field data of the type and quality given in the paper enlarge the designer's experience to a significant degree. Description of installation methods is also a contribution

[a] December, 1973, by Adel S. Saada and Chin-Der Ou (Proc. Paper 10225).

[b] December, 1973 by Kenneth R. Ware, Melvin Mirsky, and William E. Leuniz (Proc. Paper 10239).

[4] Pres., Spencer, White & Prentis, Inc., New York, N.Y.

layer was vibrated with a hand-held vibrator through a plate that was placed on top of the sand. Whether this procedure would result in isotropic specimens was the object of some attention. As explained in the paper, anisotropic behavior of the sand would be detected in the extension tests. Since the values of ϵ_1 were found to be essentially equal to the values of ϵ_2 for extension tests on both dense and loose specimens (Fig. 5), it was concluded that the sand specimens prepared for this study were isotropic. Note that the particles of Monterey No. 0 sand have subangular to subrounded grains with about equal dimensions in all directions such that they would be expected to deposit with no preferred orientations. Sands with more angular particles may have a greater tendency to adopt preferred orientations, and specimens formed of such sands therefore might be more likely to have anisotropic properties.

To obtain stress-strain data after failure, the major deviator stress was strain controlled in accordance with the suggestions of Lundgren, Mitchell, and Wilson (43). The deviator stress in the horizontal direction, however, was stress controlled, and was applied by a pressure cylinder. The major deviator stress was always applied in the vertical direction and thus there was no possibility of observing possible anisotropic effects except in the extension tests.

Daniel suggested that triaxial compression tests ($b = 0$) are fundamentally different from tests with constant values of b greater than zero, and that the rapid increase in friction angle close to triaxial compression conditions may be related to this difference. It is not clear to the writers in which way the tests are different. The stress-paths used were chosen as the simplest possible which would lead to failure with various relative magnitudes of the intermediate principal stress. Because the failure stresses were not known beforehand, it was considered by the writers to be easiest to maintain a constant ratio between the horizontal and vertical deviator stresses (expressed by parameter b) throughout each test. It has been found that the stress-path has little influence on the friction angle (7,15). Similar conclusions were made in the discussion by Reades and Green regarding triaxial extension test results.

Other points raised by Daniel concern the method of application of deviator stresses and the result of the plane strain tests. The vertical deviator stress, $\sigma_1 - \sigma_3$, was increased at a constant strain rate and the horizontal deviator stress, $\sigma_2 - \sigma_3$, was increased manually in small steps so as to maintain a constant ratio between these deviator stresses. The manual increase of the horizontal deviator stress was done using increments that were much smaller than the increments between readings. Therefore, an essentially smooth relationship between the two deviator stresses was achieved. In the two plane strain tests a complete series of points relating ϵ_1 to ϵ_3 was recorded (Fig. 5). Since all values of ϵ_2 were equal to zero in the plane strain tests, only the points corresponding to failure are shown on the upper diagrams in Fig. 5.

Goldscheider and Gudehus suggested that Eqs. 4 and 5 might be incorrect because a failure criterion in terms of stress invariants was not used for derivation of these equations. However, any failure criterion for cohesionless soil which can be expressed in terms of stress invariants can be plotted in the sin ϕ-b diagram. The Taylor's series expansion may be used to account for any shape in this diagram, and failure criteria in terms of stress invariants are therefore not excluded from consideration. Note that both the initial and the final slopes of the ϕ-b relationships, for which analytical expressions were developed (Eqs. 4 and 5), fit very well to the results presented by Reades and Green in Fig. 11.

Finally, as the result of a typographical error in Eq. 12 (see Errata), Eq. 23 derived by Goldscheider and Gudehus is not correct; Eqs. 4 and 5 do in fact define the initial and final slopes of the failure surface in a ϕ-b diagram.

Appendix.—Reference

43. Lundgren, R., Mitchell, J. K., and Wilson, J. H., "Effects of Loading Method on Triaxial Test Results," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 94, No. SM2, Proc. Paper 5844, Mar., 1968, pp. 407-419.

Errata.—The following correction should be made to the original paper:

Page 808, Eq. 12: Should read $df/d\sigma_2 = -(df_1/db)\ [1/(\sigma_1 - \sigma_3)]$ instead of $df/d\sigma_2 = -(df_1/db)\ [1/(\sigma_2 - \sigma_3)]$

Compaction of Hydraulically Placed Fills[a]

Closure by Willard J. Turnbull[5] and Charles I. Mansur,[6] Fellows, ASCE

The writers wish to thank Reitz and Hunter for their interest and comments on hydraulic fill construction, and agree with the importance of proper technical quality control during placement of hydraulic fills.

In the discussion by Reitz and Hunter, the statement was made that the field testing for control of the powerblock fill for Rush Island Plant "did not show acceptably uniform testing results for apparent constant characteristics of fill. . . ." The basis of this statement is not given. The characteristics of the sand being dredged were not constant—the D_{50} size varied from about 0.3 mm-1.0 mm, the uniformity coefficient varied from about 1.6-4.0, and the gravel content from 0%-20+%. The junior writer was responsible for field control of the powerblock fill for the Rush Island project, and is not aware of any unacceptable results of the tests made on over 200 samples to determine the relative density of the fill being placed. A maximum and minimum test was made for every in-situ density test. The method of determining the maximum and minimum densities was checked independently by another laboratory and the method of in-situ sampling and determination of maximum and minimum densities was checked by the prime consultant for the project. Special precautions were taken in the field to take in-situ samples where there was no or little

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[a] November, 1973, by Willard J. Turnbull and Charles I. Mansur (Proc. Paper 10170).

[5] Consultant, McClelland Engrs., St. Louis, Mo.
[6] Vice Pres., McClelland Engrs., St. Louis, Mo.

stratification as such can adversely affect the results of any relative density determination.

It is not clear in the discussion of the Rush Island project which fill the discussers are referring to, where they state the data reported for "this fill" was based on correlations from Standard Penetration *N* values. The powerblock fill relative densities referred to in the paper by the writers were based on individual maximum-minimum tests for each field density test, and not on any Standard Penetration *N* value. (In a subsequent personal communication with Reitz, it was learned that the reference to "this fill" was not to the Rush Island project but was intended to refer to hydraulic fills in general.)

Forty relative density tests on sand fill dredged from the Mississippi River without any supplemental compaction, but with vertical downward drainage and no ponding, for the Rush Island power-block fill showed relative densities ranging from 22% to about 60% with an average of 41%.

In the discussion by Reitz and Hunter, they state that of 125 field tests on hydrauliked sandfill receiving no mechanical compaction showed that 83% of the samples taken had relative densities in excess of 70% and that 69% of the samples had relative densities in excess of 80%. No information is presented regarding the type of fill or the method of testing. The writers in their experience have never tested hydraulic fills that had such high densities without supplemental mechanical compaction. Poulos and Hed report (22) the average relative density of four hydraulic sandfills that received no mechanical compaction to be 50% with a variation of about ±16%.

The maximum and minimum laboratory density has not been standardized and at best is erratic between laboratories. The minimum density is probably more uniformly performed than the maximum. Since the discussions do not state how the density tests for relative density comparison were performed in the laboratory, the writers would be interested in knowing how the laboratory tests were performed on the fills in the discussion described.

In the writers' opinion, it would be unwise to count on obtaining a relative density more than about 50% ±10% for hydraulic fills without supplemental mechanical compaction until proved otherwise for the particular sand being pumped and dredging procedure being used. If the sand is pumped into ponded water, or without drainage, the resulting relative density may be as low as 20%–30%.

Subsequent to the time the paper was written, the senior writer was responsible for controlling a 1,000,000-cu yd extension of the Greenville, Miss., hydraulic fill for the Mississippi Power and Light Company. This fill was to carry mostly light loads except for the fuel tank area. The fill was built with a large 30-in. dredge and initially only one heavy rubber tired roller was brought on the job. With only one roller it was difficult to get the average relative density of the fill up to 50%. By addition of a second roller the (arithemetic mean) relative density was brought up to 61%. The sand was a fine to medium uniformly graded sand with usually less than 5% of 200-mesh fines. There were obvious indications that if this fill had not been rolled, the average relative density probably would have been less than 40%.

A few months later, the same contractor with the same dredge placed a 1,500,000-cu yd sandfill for the Mississippi Power and Light Company at Vicksburg, Miss. The sand for this fill was uniformly graded and somewhat coarser than the Greenville sand. It also contained some gravel and very little 200-mesh fines. Two heavy rubber tired rollers were used on this fill full time and one caterpillar tractor a good portion of the time. An average relative density of 70% was obtained for this fill with the preceding compactive effort. A few samples taken in small areas which were not rolled had a relative density of only about 50%.

Appendix.—Reference

22. Poulos, S. J., and Hed, A., "Density Measurements in a Hydraulic Fill," *STP 523*, American Society for Testing and Materials, July, 1973, p. 402.

TIEBACK WALL CONSTRUCTION—RESULTS AND CONTROLS[a]

Closure by Kenneth R. Ware,[5] Melvin Mirsky,[6] and William E. Leuniz,[7] Members, ASCE

The writers wish to thank White for his comments concerning the use of soil anchor tiebacks. White is correct in saying that it is an error to show grout extending tight up against the timber lagging. Fig. 4 was intended to be a sketch rather than a detailed explanation of the tieback installations. However, in most cases the grout did extend above the influence line. In the writers' experience this is customary with most tieback installations. White's company, Spencer, White & Prentis, Inc., is an exception because their installation procedure incorporates a method of removing the grout above the influence line.

Correlation of the failure rate with the soil type in the anchorage zone was found to be unreliable because of the erratic nature of soil stratification at the project site; analyses of data from this study was not included in the paper. We did find a direct correlation between the failure rate and the quantity of group pumped into each anchorage zone for all soil types; and this was the basis of the recommendation (Item 2*d* on p. 1151) that the volume of grout inject in the anchor zone of each tieback should be recorded.

The "toggle" on the end of the tendons was intended to prevent them from pulling out as the casing was removed. Note that in Method 2 (part 1, p. 1142) some difficulty was experienced despite the use of the "toggle."

[a]December, 1973, by Kenneth R. Ware, Melvin Mirsky, and William E. Leuniz (Proc. Paper 10239).

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