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Triaxial Shear Characteristics of Clayey Gravel Soils

Caractéristiques de la résistance au cisaillement triaxial des sols graveleux argileux

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Summary

During recent years, the Bureau of Reclamation's Earth Laboratory has been studying all of the physical properties of gravelly soils because these soils are used extensively in Bureau earth dams, canal embankments, and earth canal linings, and because prior soils research has been largely limited to testing the finer soil fraction. This paper was prepared to present the results of recent research tests for studying the shear characteristics of impervious gravel soils which contained varying amounts of a clayey soil matrix.

Large-scale triaxial testing equipment was used for this study on the clayey gravel soils. The testing program was formulated to evaluate the effect of gravel content on shear strength for materials containing particles up to 3 inches in size. The shear test results are presented on the basis of effective stresses and pertinent information on positive and negative pore pressures measured during the tests is given. Also included in the paper is a very brief review of the previously reported shear tests made on pervious sandy gravels, for the purpose of comparing the shear properties of the pervious materials with those of the impervious clayey gravels. The conclusions reported in this paper are based on the analysis of 20 individual shear specimens of the clayey gravel soils.

Introduction

In most past and current research studies of soils, the laboratory tests are made on the finer, soil-matrix fraction of the total material from which the gravel particles have been removed. As a general rule, tests are seldom made on soils with particles larger than the 3/16-inch (No. 4 sieve) size, because large specimens and large testing equipment are required and testing costs are increased greatly. The need for information on the physical properties of the whole material, containing both the fine and gravel fractions, was recognized by the Bureau of Reclamation because so many of the materials used in our earth dams, earth canal embankments, and earth canal linings contain significant amounts of gravel (+ 3/16-inch particle size). It is often good engineering practice to use gravelly soils in earthworks because of the high stability, low settlement, and high erosion resistance of these materials.

For these reasons, an extensive research program was formulated by which all of the properties of gravelly soils will eventually be studied. This program is being pursued as time and funds are available. Some of the studies have been completed and have been reported. These include studies on the compaction characteristics of sandy, silty, and clayey gravels (HOLTZ and LOWITZ, 1957), permeability-consolidation characteristics of pervious sandy gravels (JONES, 1954), the consolidation and pore pressure characteristics of impervious

Sommaire

Dans ces dernières années le Laboratoire d'Etude du Sol du Bureau of Reclamation a étudié très complètement les propriétés physiques des sols graveleux ; ces sols sont utilisés d'une manière étendue dans les barrages en terre, dans les remblais et les revêtements en terre des canaux ; d'autre part les recherches antérieures ont été en général limitées à la fraction la plus fine du sol.

Ce rapport a été préparé pour présenter les résultats de recherches récentes faites pour étudier la résistance au cisaillement des sols graveleux imperméables contenant une quantité variable de matériaux argileux.

Un équipement d'essai triaxial de grande dimension a été utilisé dans cette étude. Le programme des essais a été formulé pour évaluer l'effet de la proportion de gravier sur la résistance au cisaillement des matériaux contenant des éléments jusqu'à 7.5 cm de diamètre. Les résultats des essais de cisaillement sont présentés sur la base des contraintes effectives et des indications sont données sur les pressions interstitielles positives ou négatives mesurées au moment des essais.

Un résumé des essais de cisaillement faits sur les graviers sableux perméables pour comparer les propriétés de cisaillement des matériaux perméables avec ceux des matériaux graviers argileux imperméables est aussi inclus dans ce rapport. Les conclusions citées sont basées sur l'analyse des essais de cisaillement de 20 échantillons de graviers argileux.

gravelly soils (GIBBS, 1950), and the shear characteristics of pervious gravelly soils (HOLTZ and GIBBS, 1956).

In the series of tests reported herein, the relationship of shear strength of clayey soils with variable gravel contents was studied. The shear test data are reported in terms of effective stresses where the effective stress (σ') is equal to the total stress (σ), less the pore pressure (u) measured during the test.

Test equipment and procedures

All shear specimens in this research study were 9 inches in diameter by 22.5 inches in length. The details of the testing apparatus have been described previously (HOLTZ and GIBBS, 1956), (NOELL, 1953), (U.S.B.R., 1960), and are discussed here only in a brief manner as necessary for an understanding of the data presented.

Fig. 1 shows the large shear testing equipment. The compression machine for applying the axial load is equipped with automatic stress-strain recorder and rate of stress and rate of strain indicators. The pressure chamber was designed for a maximum working lateral pressure of 150 psi. It consists of aluminum top and bottom plates, aluminum or clear plastic chamber cylinders of 14-inch-inside diameter, and six 1-inch stainless-steel bolts to hold the assembly together.

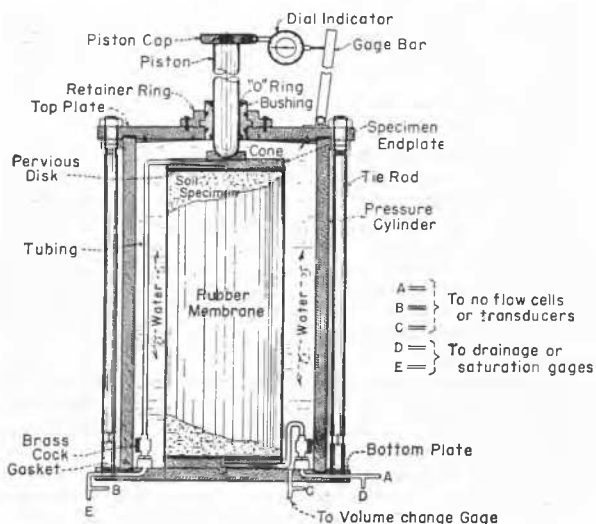
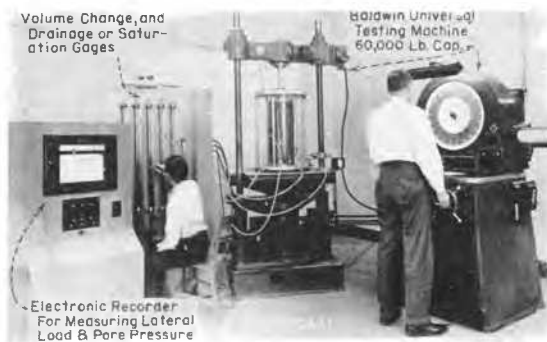


Fig. 1 Triaxial shear equipment for large specimens.
 Equipement de cisaillement triaxial pour de grands échantillons.

The piston through which the axial load is applied to the specimen was made of stainless steel 2 inches in diameter. The piston passes through a bushing in the top chamber plate, and "O" rings recessed into the bushing provide a positive seal around the piston. A small stainless steel plate attached to the top of the piston provides a bearing plate for the head of the compression machine. A dial indicator of 1-inch travel is attached to this plate. The indicator travels along a slanting bar, attached to the chamber top, as the specimen is compressed. By this means, axial deformation can be measured for as much as 7 inches or about 30 per cent axial strain.

The specimen end plates were made from 1-inch aluminum plate and recessed on the specimen side for inserting 0.125-inch-thick plates perforated with 1/32-inch-diameter holes, or for inserting very fine ceramic disks. The ceramic disks are used when both positive and negative pore pressures are to be measured. Porous ceramic inserts are also available for measuring pore pressures in the inner portion of specimens. The top and bottom end plates and inserts are fitted with plastic tubing and brass cocks for connecting to drainage or saturation tanks and pore pressure cells. Plastic tubing made of a vinylidene chloride polymer is used because of its flexibility and ability to stand 150-psi outside pressure without collapsing. The rubber membranes for enclosing the soil specimens are 9 inches in diameter, 24 inches long, and 0.06 inch thick. Stainless steel worm-drive hose clamps are used to clamp the membrane to the end plates. Tests have shown that membranes of this thickness have very little effect on the strength of the specimens (U.S.B.R., 1960).

The lateral pressure is applied by means of air pressure on the water-filled chamber. The pore pressures at each end of the specimen and the lateral pressure are measured either by no-flow pressure cells or automatically through pressure transducers by means of an electronic strip recorder. Water gages with Vernier reading glasses are mounted on the back side of the control panel for reading the volume change of the specimen in terms of the amount of water expelled or taken up by the chamber during the test.

The above facilities allow the measurement of the following data during the test : axial load, axial strain, volume of specimen, volume change of specimen, and pore pressure.

All of the tests were performed using a constant chamber pressure (σ_3) and in a sealed condition ; that is, no drainage was allowed at any time. After each specimen was prepared, it was immediately placed in the rubber membrane which was clamped to the end plates. After being assembled in the testing chamber, the chamber pressure was applied and volume change and pore pressure were allowed to reach equilibrium prior to the application of the deviator load.

The rate of axial load application was controlled to a speed which would provide a correct relationship between the pore pressures and volume changes measured (ELLIS and HOLTZ, 1959). This relationship was plotted, as shown in Part *d* of Fig. 4, continuously as the test progressed. It was found that an axial strain rate of 0.02 inches per minute (0.09 per cent per minute) provided satisfactory results and this rate was used throughout the test series.

Materials

The gradation of the various soil-gravel mixtures selected for this study are shown in Fig. 2, and were selected as repre-

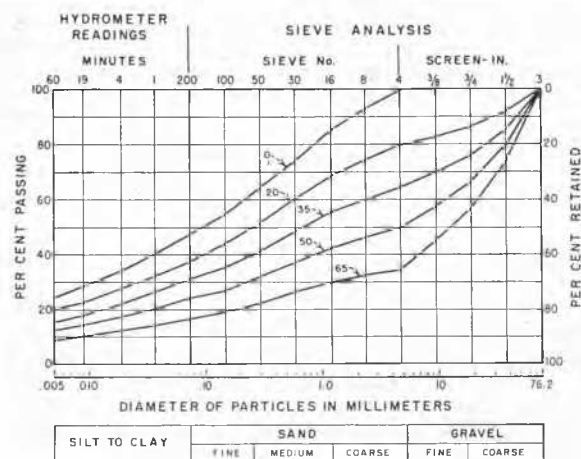


Fig. 2 Gradation of clayey gravel soils.
 Granulométrie des sols graveleux argileux.

sending common impervious materials used in our earth embankment structures. The soil fraction (fraction smaller than 3/16-inch size) is classified as a SC-CL (Unified Classification System) (WAGNER, 1957). The fraction passing the No. 40 sieve had a $L_L = 49$ and a $P_I = 28$. To the soil fraction were added varying amounts of 3/16 to 3-inch gravel to produce the gradations shown in Fig. 2. The gravel particles were predominantly gneiss, granite, and schist, obtained from a river deposit. The particles were of subangular to subrounded shape. The specific gravity of the clay and clay-gravel mixtures varied from 2.66 to 2.70. As the material was reused numerous times, a gradation test was made after completion of the tests to check particle breakdown. It was found that particle size changes were very minor.

Specimen preparation

The materials used in this study were the same as the clayey gravels used in the large-scale compaction tests to determine the compaction characteristics of gravelly soils, where standard Proctor compactive effort (12,375-foot-pounds per cubic foot) was applied (HOLTZ and LOWITZ, 1957). Therefore, the dry density conditions were controlled to 95 per cent of the values obtained in that study. Moisture conditions were controlled to 1 per cent less than optimum moisture for the total material. Fig. 3 (solid lines) shows the compaction

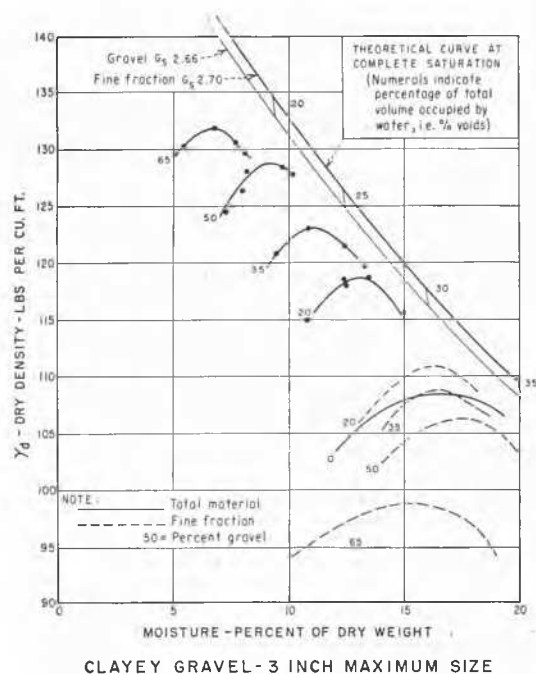


Fig. 3 Compaction characteristics of clayey gravel soils.
Caractéristiques de compactage des sols graveleux argileux.

characteristics of the soils having different amounts of gravel. In this figure, the solid lines show the compaction characteristics of the total materials. The dashed lines show the compaction characteristics of the fine fractions (smaller than 3/16 inch) contained in the total materials.

Each material was brought to its optimum moisture condition and allowed to "season" for a minimum period of 48 hours. The material was then compacted to desired density in a hinged, three-section mold of 9-inch diameter by 22.5 inches long (inside dimensions). Molding was performed by compacting the gravelly material in seven equal layers by a drop hammer. Each layer was lightly scarified prior to placing the succeeding layer. Immediately after remolding, the specimens were removed from the mold, the end plates placed on them, and the rubber sleeves slipped over the specimens and clamped to the end plates.

Effect of gravel content

The effect of gravel content on the shear strength of clayey soils was the principal objective of this series of research tests. The typical variations in shear strength, with gravel contents varying from zero to 65 per cent for the lean clay and river gravel mixtures, are shown by the shear strength envelope plots of Figs. 4 through 8. Included at the top of each figure are pertinent test data for each specimen tested.

Part c of Fig. 4 shows the pore-pressure — axial-strain relationship for Specimen 4 of the test series on material containing no gravel in which pore pressures were measured simultaneously with perforated and ceramic end plates and a central porous insert. A brief summary of the results of the tests, with the computed friction and cohesion values for each material is given in Table 1.

Table 1
Summary of shear test data-clayey gravel soil
Sommaire des données des essais de cisaillement
Sols graveleux argileux

Per cent gravel	Per cent compaction at Proctor effort	Average dry density (pcf)	Average moisture (per cent)	Computed shear values	
				Tan Φ'	\bar{c} (psi)
0	95	103.0	15.8	0.45	8.7
20	95	112.8	11.8	0.48	7.0
35	95	116.0	10.3	0.47	8.3
50	95	122.7	8.3	0.63	4.5
65	95	125.4	6.2	0.68	5.0

The shear test data plots given in Fig. 4b and Figs. 5 through 8 are recorded in terms of vector curves, rather than conventional stress circles which show only the failure condition for each shear specimen of a test series. These vector curves are drawn through computed points of the effective normal stresses and shear stresses on the failure planes at periodic intervals throughout the shear tests. By this means of plotting the test data, the relationship between the normal and shear stresses can be seen throughout the test and the conditions at failure can be studied more effectively.

The envelopes of limiting shear resistance shown on Figs. 4 through 8 were drawn as straight lines. The straight line interpretation appeared to provide the most logical interpretation for the entire series of tests. The "coefficient of friction" ($\tan \Phi'$) and "cohesion" (\bar{c} intercept) values shown on Table 1 are called computed shear values and were obtained from the straight line envelope. The point on each vector

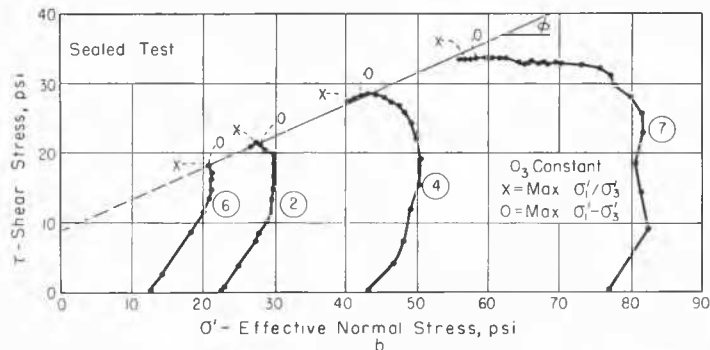
curve where maximum $\frac{\sigma'_1}{\sigma'_3}$ conditions (HOLTZ, 1947) existed

was taken as the failure point of that specimen. The envelopes were computed as the most probable straight lines through these failure points by the least squares method. The points of maximum $\sigma'_1 - \sigma'_3$ conditions are also shown on the vector plots.

The curvature of the moisture-density plots of the compaction test data (Fig. 3) for the clay-gravel mixtures was great. Therefore, small changes in placement moisture gave pronounced changes in placement density, which made density control difficult. Because the compaction curves at the moisture-density conditions selected were close to the 100-per cent saturation line, the development of considerable pore pressure resulted during many of the tests. This was particularly true at higher loadings where significant volume changes took place. The development of high pore pressures resulted in reducing the range of effective stresses for any test series and also caused considerable prestressing as evidenced by sharp back-hooking of the vector plots at higher loadings. (As an example, see Spec. 7, Fig. 4.)

The results given on Figs. 4 through 8 and summarized in Table 1 show the effect of varying amounts of gravel in the clay soil matrix. The coefficient of friction, $\tan \Phi'$, of the soil with no gravel was 0.45 and the cohesion 8.7 psi. The friction increased and the cohesion decreased only small

Specimen No.	Initial Specimen Data			Test Values at Failure Max σ'_1/σ'_3				Shear Values Corrected for Pore Pressure	
	Dry Density (lbs. per cu. ft.)	Water Content (per cent)	Degree of Saturation (per cent)	Applied Lateral Press. (psi.) σ'_3	Effective Lateral Press. (psi.) σ'_3	Volume Change (per cent of Initial)	Deviator Stress (psi.) $\sigma'_1 - \sigma'_3$	Tan ϕ'	Cohesion c' (psi.)
6	103.0	15.8	67	12.5	7.9	4.59	37.7		
2	102.2	15.8	66	25.0	11.8	7.61	44.6		
4	103.3	15.8	68	50.0	20.2	10.16	57.7		
7	103.3	15.8	68	100.0	31.2	11.27	70.2	0.45	8.7



LABORATORY RESEARCH

Sandy Clay - Lean Clay (SC-CL)

Sample No. 27C-2

Compacted

Large Scale Triaxial Shear

Specimen Size 9"x22 1/2"

Per cent Gravel 0

Per cent Sand 50

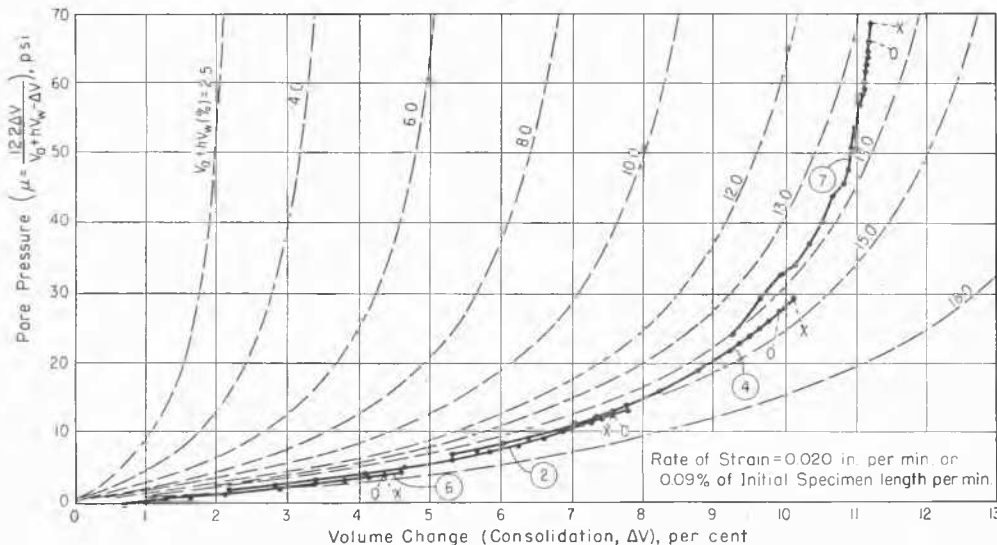
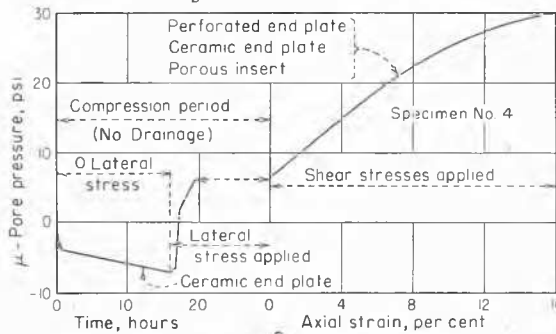


Fig. 4 Detailed shear test data — 0 per cent gravel.

Données détaillées de l'essai de cisaillement — Pas de gravier.

amounts when 20 and 35 per cent gravel was added, showing that there was very little large particle interference and, therefore, the soil had shear properties similar to the clay matrix. However, when the gravel content was raised to 50 per cent, the effect of the gravel was readily apparent and the $\tan \phi'$ value was increased to 0.63. As additional

gravel was added to provide 65 per cent, the $\tan \phi'$ increased to 0.68. A corresponding reduction in the cohesion of the soil was apparent as it became more granular. Thus, it can be stated that, as the gravel content of a clay soil is increased, a rather abrupt change in shear strength can be expected when sufficient gravel is added to provide large particle

Specimen No.	Initial Specimen Data			Test Values at Failure. Max. σ'_1/σ'_3				Shear Values Corrected for Pore Pressure	
	Dry Density (lbs. per cu. ft.)	Water Content (per cent)	Degree of Saturation (per cent)	Applied Lateral Press.(psi) σ_3	Effective Lateral Press.(psi) σ'_3	Volume Change (per cent of Initial)	Deviator Stress (psi) $\sigma'_1 - \sigma'_3$	Tan ϕ'	Cohesion c' (psi)
7	113.1	11.8	65	6.2	4.2	-1.46	26.3	(Indicated from Max. $\frac{\sigma'_1}{\sigma'_3}$)	
8	112.7	12.0	65	18.0	8.5	-5.87	32.9		
5	113.2	11.4	64	37.5	19.1	-7.94	51.7		
6	112.4	12.1	66	50.0	23.4	-9.21	54.0	0.48	7.0

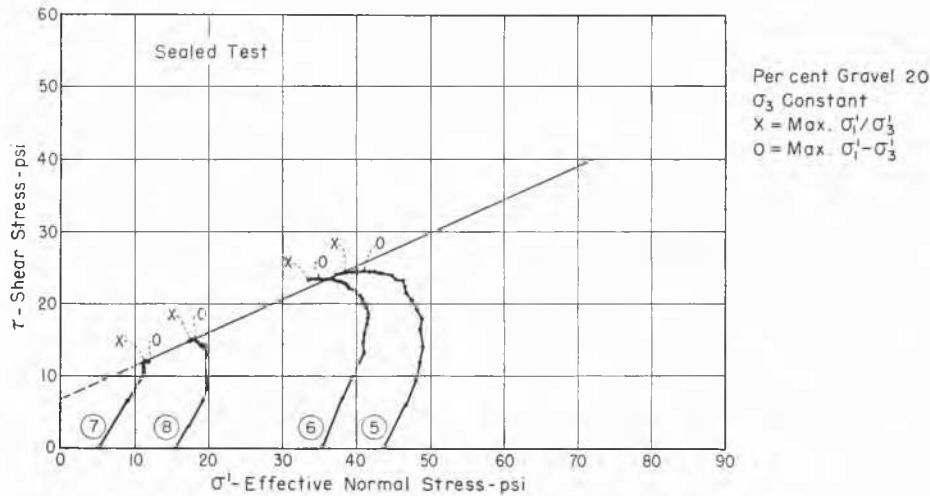


Fig. 5 Shear test results — 20 per cent gravel.
R  sultats de l'essai de cisaillement — 20 pour cent du gravier.

Specimen No.	Initial Specimen Data			Test Values at Failure. Max. σ'_1/σ'_3				Shear Values Corrected for Pore Pressure	
	Dry Density (lbs. per cu. ft.)	Water Content (per cent)	Degree of Saturation (per cent)	Applied Lateral Press.(psi) σ_3	Effective Lateral Press.(psi) σ'_3	Volume Change (per cent of Initial)	Deviator Stress (psi) $\sigma'_1 - \sigma'_3$	Tan ϕ'	Cohesion c' (psi.)
1	115.2	11.3	67	12.5	7.6	-3.82	39.9	(Indicated from Max. $\frac{\sigma'_1}{\sigma'_3}$)	
2	116.0	10.3	63	25.0	14.4	-6.33	51.5		
4	116.0	10.0	61	75.0	40.8	-10.27	83.5		
5	116.8	9.6	59	75.0	39.0	-10.05	91.6	47	8.3

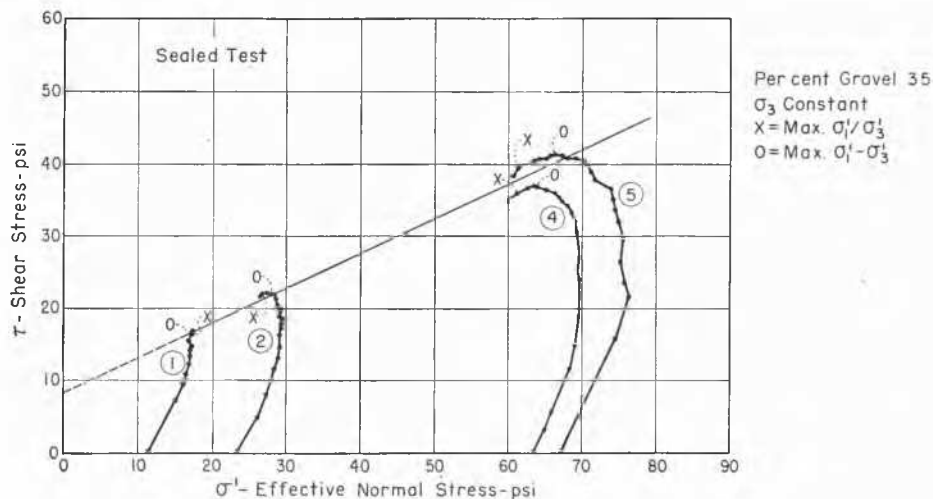


Fig. 6 Shear test results — 35 per cent gravel.
R  sultats de l'essai de cisaillement — 35 pour cent de gravier.

Specimen No.	Initial Specimen Data			Test Values at Failure Max σ'_1/σ'_3				Shear Values Corrected for Pore Pressure	
	Dry Density (lbs per cu. ft.)	Water Content (per cent)	Degree of Saturation (per cent)	Applied Lateral Press.(psi) σ_3	Effective Lateral Press.(psi) σ'_3	Volume Change. (per cent of Initial)	Deviator Stress (psi) $\sigma'_1 - \sigma'_3$	Tan ϕ'	Cohesion c' (psi)
1	121.2	9.1	65	12.5	4.1	-5.18	25.8	(Indicated from Max. $\frac{\sigma'_1}{\sigma'_3}$)	
2	122.5	8.7	67	25.0	13.3	-8.28	43.4		
4	123.7	8.8	68	75.0	20.0	-10.95	56.3		
5	123.5	6.7	51	100.0	55.3	-14.42	137.9		
								63	4.5

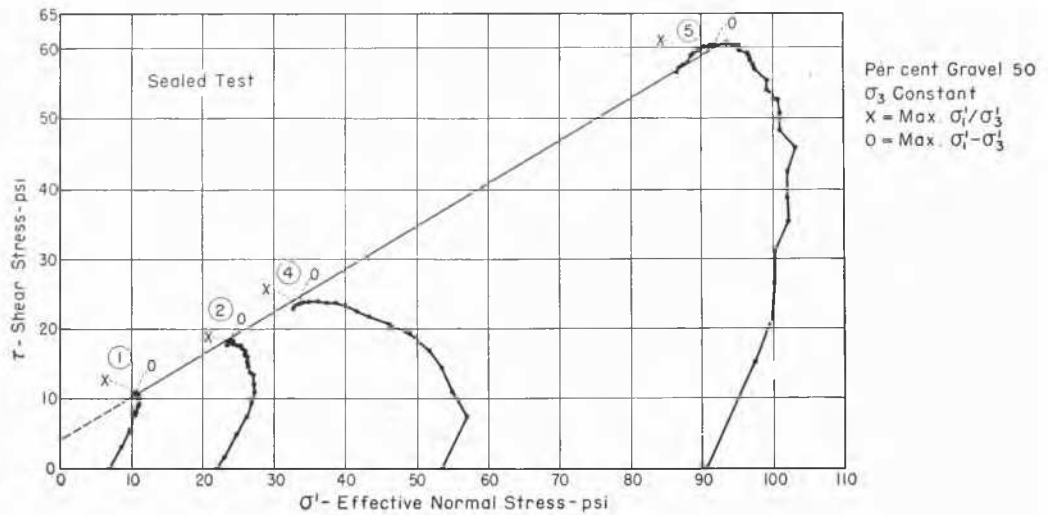


Fig. 7 Shear test results — 50 per cent gravel.
Résultats de l'essai de cisaillement — 50 pour cent de gravier.

Specimen No.	Initial Specimen Data			Test Values at Failure. Max. σ'_1/σ'_3				Shear Values Corrected for Pore Pressure	
	Dry Density (lbs. per cu. ft.)	Water Content (per cent)	Degree of Saturation (per cent)	Applied Lateral Press. (psi)	Effective Lateral Press. (psi)	Volume Change (per cent of Initial)	Deviator Stress (psi)	Tan ϕ'	Cohesion c' (psi)
				σ_3	σ'_3		$\sigma'_1 - \sigma'_3$		
				(Indicated from Max. $\frac{\sigma'_1}{\sigma'_3}$)					
1	125.5	6.6	54	12.5	8.6	-2.80	44.2		
2	126.0	6.3	52	25.0	20.2	-5.41	67.5		
3	124.3	6.2	49	40.0	30.7	-8.07	97.6		
6	126.0	5.9	49	50.0	34.3	-7.37	109.8	0.68	5.0

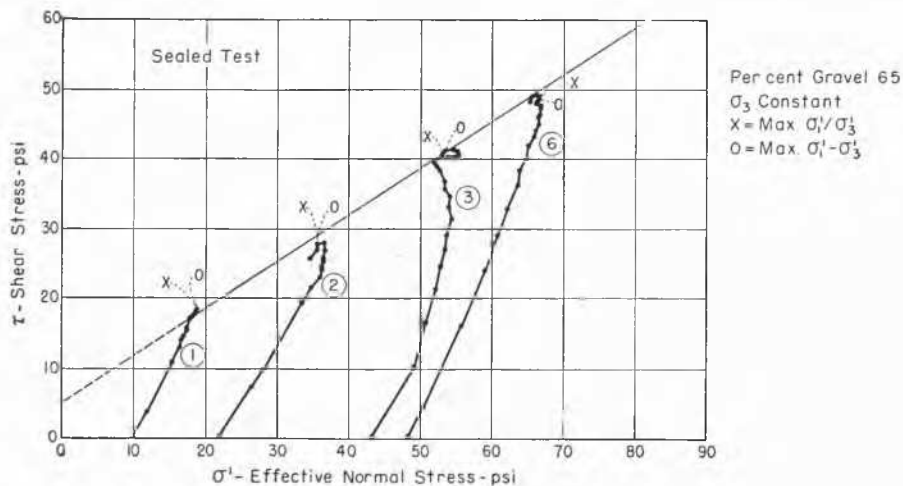


Fig. 8 Shear test results — 65 per cent gravel.
Résultats de l'essai de cisaillement — 65 pour cent de gravier.

interference. The per cent gravel content at which this occurs would probably vary with the clay matrix and gravel types. This characteristic of large particle interference has been observed in the study of other properties of gravelly soils. Volume strains decreased as the large particles replaced compressible soil, as would be expected.

It is of interest to compare the shear characteristics of these clayey gravel soils with those previously obtained for sandy gravel soils (HOLTZ and GIBBS, 1956). The results of those tests, for 3-inch-maximum size gravel only, are summarized briefly in Table 2. Fig. 9 shows the gradation

Table 2

Summary of Shear test data — Sandy gravel soil
Sommaire des données des essais de cisaillement
Sols graveleux sableux

Per cent gravel	Per cent compaction at Proctor effort	Average dry density (pcf)	Computed shear values	
			Tan Φ	\bar{c} (psi)
0	93.3	109.5	0.72	3.7
20	93.3	116.5	0.79	6.6
50	92.5	125.4	0.88	5.5
65	92.2	129.9	0.90	5.0

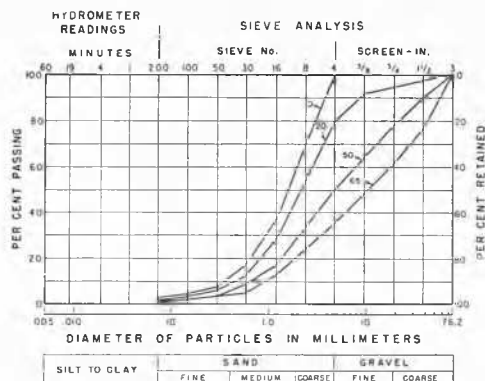


Fig. 9 Gradation of sandy gravel soils.
Granulométrie des graviers sableux.

characteristics of the sandy soils. The sand and gravel were obtained from the same river deposit as the gravel used in the clayey gravel tests. The shear strength envelopes for the sandy gravel and clayey gravel soils are shown on Figs. 10a and 10b, respectively, for comparative purposes. The sandy gravel soils of 3-inch-maximum size were tested at an average density equivalent to 93 per cent of compaction at Proctor compactive effort, which was equivalent to about 70 per cent relative density. Under these conditions, the tan Φ' friction values varied from 0.72 for zero gravel content to 0.90 for 65 per cent gravel content, systematically increasing as the gravel content increased with a leveling off at the higher gravel contents. Indicated cohesive strength (primarily due to particle interlocking) was in the order of 4- to 6-psi.

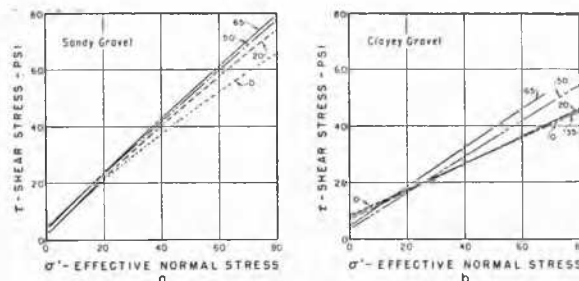


Fig. 10 Summary of shear test results for sandy and clayey gravel soils.

Sommaire des résultats des essais de cisaillement pour les sols graveleux sableux et argileux.

While the shapes of the shear strength envelopes for the compacted cohesive soils were practically straight lines, there was a definite slight curvature to the shear strength envelopes of the sandy gravel soils. This was due to the pronounced particle interlocking which appeared to affect the shear strength of these soils and was particularly noticeable at the lower lateral pressures. This effect was not noticeable in the clayey gravel soils.

In summary, it may be said that the clayey gravel soils had lower frictional strength than the sandy gravel soils for equal gravel contents, the reduction in tan Φ' values being in the order of 0.22 to 0.27, or from 8 to 12 degrees in the friction angle.

References

- [1] ELLIS, WILLARD and HOLTZ, W. G. (1959). A Method for Adjusting Strain-rates to Obtain Pore Pressure Measurements in Triaxial Shear Tests, A.S.T.M. *Symposium on Time Rates of Loading in Soil Testing*.
- [2] GIBBS, HAROLD J. (1950). The Effect of Rock Content and Placement Density on Consolidation and Related Pore Pressure in Embankment Construction, *Proceedings of the A.S.T.M.* Vol. 50, p. 1343.
- [3] HOLTZ, W. G. (1947). The Use of the Maximum Principal Stress Ratio as the Failure Criterion in Evaluating Triaxial Shear Tests on Earth Materials, *Proceedings of the A.S.T.M.* Vol. 47.
- [4] — WESLEY G. and GIBBS, HAROLD J. (Jan. 1956). Triaxial Shear Tests on Pervious Gravelly Soils. A.S.C.E. *Journal of the Soil Mechanics and Foundations Division*, Paper No. 867.
- [5] — and LOWITZ, CLEMITH A. (Dec. 1957). Compaction Characteristics of Gravelly Soils. Conference papers of the Joint Meeting of A.S.T.M. Committee D-18 and the Sociedad Mexicana de Suelos, A.S.T.M. *Special Technical Publication* No. 232, p. 67.
- [6] JONES, CHESTER W. (1954). The Permeability and Settlement of Laboratory Specimens of Sand and Sand-Gravel Mixtures, *Symposium on Permeability of Soils*, A.S.T.M. *Special Technical Publication* No. 163, p. 68.
- [7] NOELL, O. A. (1953). Triaxial Shear Equipment for Gravelly Soils, *Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering*, vol. I, p. 165.
- [8] U.S. Bureau of Reclamation (1960). Shear Strength of Cohesive Soils, *Proceedings of the A.S.C.E. Research Conference on the Shear Strength of Cohesive Soils*.
- [9] WAGNER, A. A. (1957). The Use of the Unified Classification System by the Bureau of Reclamation, *Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering*, vol. I, p. 125.