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JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION

Tjipanundjang Dam in West Java, Indonesia

By Laurence D. Wesley,[1] M. ASCE

Introduction

The Tjipanundjang Dam is a homogeneous earth dam located on a plateau about 30 km south of the city of Bandung in West Java, Indonesia. It has a height of 34 m and was constructed between 1928 and 1939. The dam is built on the Tjisangkuj River not far from the point where the river leaves the Pangalengan plateau and descends fairly rapidly towards the city of Bandung. The dam forms the upper of two lakes which provide storage for hydroelectric generating stations some distance below the lakes. The lower lake was formed by the construction of two earlier dams built in the early 1920's. These earlier dams were built of similar soil to the Tjipanundjang Dam but were only about half its height.

The design and construction of the dam was originally described by Beckman (1), De Vos (2), and Van Es (10). In recent times the dam has been reexamined by Terzaghi (8) in connection with the construction of the Sasumua Dam, in Kenja. In particular, the mineralogical and chemical composition of the soil used for the dam was investigated and apparently shown to be similar to the Sasumua Dam material. Dixon and Robertson (3) have also commented on the Tjipanundjang Dam.

The purpose of this paper is to provide further information on this rather interesting dam and to suggest that the very unusual properties of the dam soil cannot be explained by a simple hypothesis such as that put foward by Terzaghi (8).

Geology and Soil Conditions at Site

The Pangalengan plateau is a small, fairly flat plateau surrounded by low volcanic peaks. The average elevation of the plateau is about 1,500 m and

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the highest peak in the area rises to 2,300 m. The volcanic activity which formed the plateau dates from the end of the Tertiary Era, when a period of volcanic activity began which formed the many volcanic peaks for which Java is famous. This volcanic activity has continued intermittently to the present day and minor geothermal activity in the form of hot springs and steam fumeroles is still apparent in several places around the Pangalengan plateau. However, no major activity has taken place in recent times in this area. Most of the plateau consists of deep layers of ash and tuff. According to De Vos and Beckman the Tjipanundjang Dam is built on tuff layers which have undergone extensive weathering to a great depth. De Vos states that a borehole put down to 25 m as part of the site investigation showed weathered tuff over the full depth of the hole. The weathered tuff is referred to by the Dutch engineers as "tuff clay." It may be more correct to describe the original material here as ash rather than tuff, as it was probably loose and uncemented. Some of it is no doubt wind deposited material but much of it is probably made up of ash deposits washed down from the surrounding slopes to form the plateau itself.

There does not appear to be any evidence to suggest that the soil is derived from basic lavas as mentioned by Terzaghi (8). There are no rock outcrops of any kind in the vicinity of the dam. As no boreholes deeper than 25 m were put down the depth of soil at the site is not known. However, the valley in which the dam is built is well over 30 m deep, and if the borehole mentioned by De Vos was carried out in the bottom of the valley as seems probable, then the depth of soil may be considerably more than 50 m.

The "tuff clay" is a yellowish brown clay belonging to a soil group known pedalogically as andosol. The term andosol (or ando soil) actually originates from Japan (meaning dark soil) and was introduced as a "great soil group" by Thorp and Smith (9). The "dark soil" refers to the upper layer of these soils which is often dark grey to black due to a high organic content. However the lower layers, at least in Indonesia, are typically light in color, varying between brown and yellowish brown. Andosols in Indonesia are derived from the weathering of volcanic material, mainly ash layers, and normally occur at altitudes above 1,200 m. The soil at the Tjipanundjang Dam is therefore not a red clay as stated by Terzaghi (8) and Dixon and Robertson (3), although these andosols are closely related to the typical tropical red clays (latosols) found in Indonesia at lower altitudes. Andosols and latosols occur very widely in both Java and Sumatra and on typical volcano slopes the latosols are found up to an altitude approaching 1,000 m. Generally before the 1,000-m altitude is reached the red color is no longer dominant and there is a general transition until above 1,100 m or 1,200 m only brown and yellowish brown soil is found. These yellowish brown soils are indeed rather unusual as will be shown later. They are not nearly as plastic and sticky as the red soils although they are clearly very fine grained. When pressed between the fingers they feel quite greasy or soapy. They possess remarkably good engineering properties.

Description of Dam

The Tjipanundjang Dam is built in a narrow curved valley, upstream from which is a natural depression forming a large storage reservoir. A sketch plan showing the layout of the dam and the borrow pit areas is shown in Fig. 1

and a cross section of the dam is shown in Fig. 2. The dam was planned to be built in two stages, the second stage being 6 m higher than the first stage. These stages are shown in Fig. 2. The first stage was completed in 1929 and the second about 10 yr later. De Vos (2) describes the design of both stages so that Terzaghi (8) is not strictly correct in stating that because of the dams satisfactory performance it was decided to raise it by 20 ft 10 yr

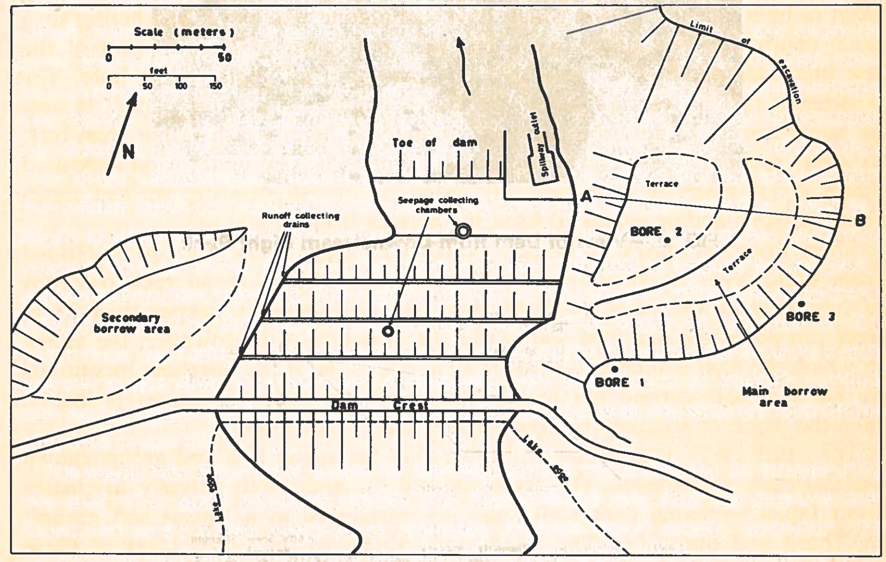

FIG. 1.—Layout of Dam and Borrow Areas

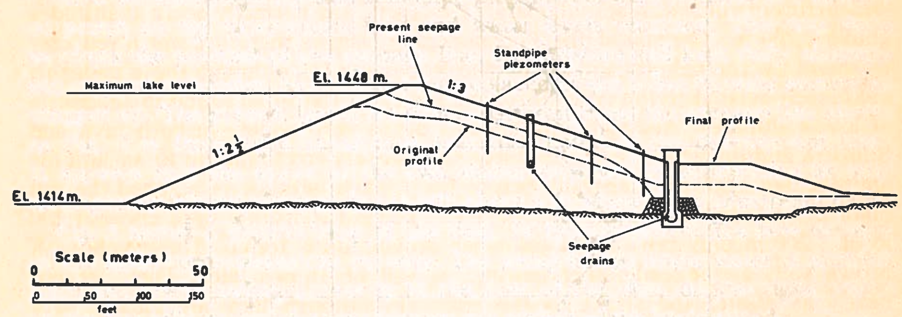

FIG. 2.—Cross Section of Dam

later. A report by Dixon and Robertson (3) that the dam was raised by 33 ft in the late 1940's appears to be without foundation. The dam has not been altered since the completion of the second stage before 1940.

Several points mentioned by De Vos and Beckman in describing the design and execution of the dam are of considerable interest. First, the homogeneous cross section was selected because there was no other material, rock, gravel,

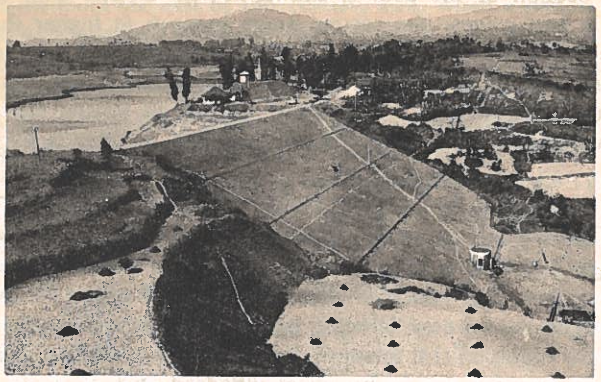

FIG. 3.—View of Dam from Downstream Right Bank

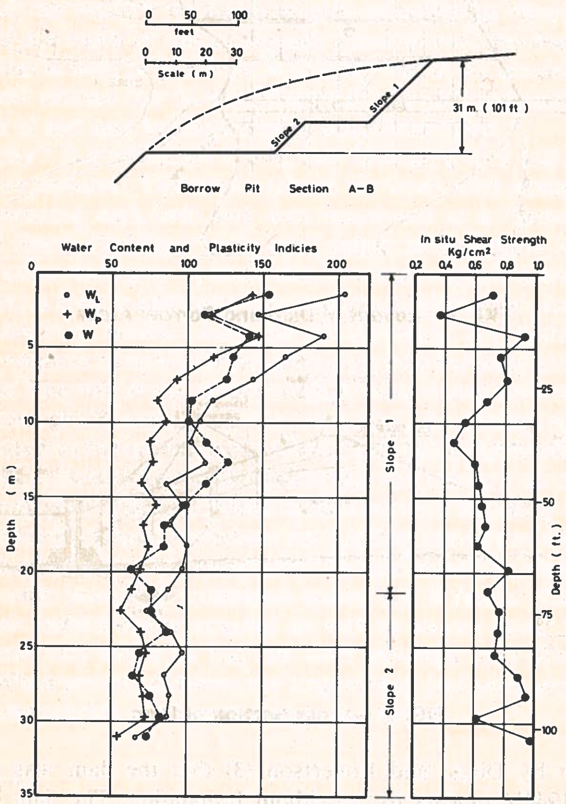

FIG. 4.—Atterberg Limits, Water Content, and In Situ Shear Strength from Borrow Pit Slopes

or sand available in the area. Mention is made of rock for the spillway tower and culverts being carried some of the way on rafts across the lower lake to reduce transport costs. Secondly, great emphasis was placed on installing suitable drains in the body of the dam to ensure the seepage line remained a satisfactory distance below the downstream slope. The position of these drains is shown in Fig. 2. When the lower lake is full the toe of the Tjipanundjang Dam is actually submerged so that the lower seepage drain actually collects seepage water from both sides and pumping is necessary to remove the seepage water from this drain. Thirdly, the criteria used for selecting the fill material is of interest. Much importance was attached by Dutch engineers of that time to a property termed the "surplus". This was defined as the difference between the "sticky" limit and the liquid limit of the soil. The "sticky" limit was determined by drawing a metal surface over the surface of a soil pat in the laboratory. The water content at which the soil started to stick to the surface was termed the sticky limit. It was believed that a soil with a positive "surplus" was not likely to slide whereas a soil with a negative surplus would easily slide. Beckman reports that in some areas at the dam site the soil had a negative "surplus" and was therefore rejected. The significance or validity of this criteria is considered briefly later. De Vos also mentions the removal of some of the soil at the surface on the valley walls because it was "laterized". Whether this is technically correct is not clear but there are occasional very thin layers of slightly reddish soil and it appears that for some reason this material was considered unsuitable and was removed.

Terzaghi has described some of the difficulties encountered in compacting the soil with steel wheeled rollers and has pointed out that no control was exercised over water content or density. Checks were made, however, by excavating test pits, to ensure that no cavities existed in the compacted soil. Note also that greater compaction was given to the soil near the upstream face in an attept to obtain lower permeability in this part of the dam.

After completion of the dam standpipe piezometers were installed at a number of points on the downstream face and water levels recorded. This was done to check the position of the seepage line. Records are still kept daily although the original standpipes have since been replaced as the tubes became somewhat clogged up. The present levels are somewhat erratic and the seepage line shown in Fig. 2 is from three standpipes near the center of the downstream face. It appears that the upper of the two drains may not be functioning as well as originally but the seepage line is still about 4 m below the downstream face.

Fig. 3 shows a general view of the dam. In the foreground the two terraces of the main borrow area can be seen. The pump house where water is pumped from the lower seepage drain is at the foot of the dam on the right. Some of the low hills surrounding the plateau are visible in the background.

Properties of Dam Material

The properties of the soil used for the construction of the dam have been studied by taking samples from the main borrow pit. To obtain some general data on the material and especially on variations in properties with depth, samples were first obtained at 2-m intervals down the slopes of the main borrow area. A cross section, A-B, of this borrow area is shown in Fig. 4. Samples were

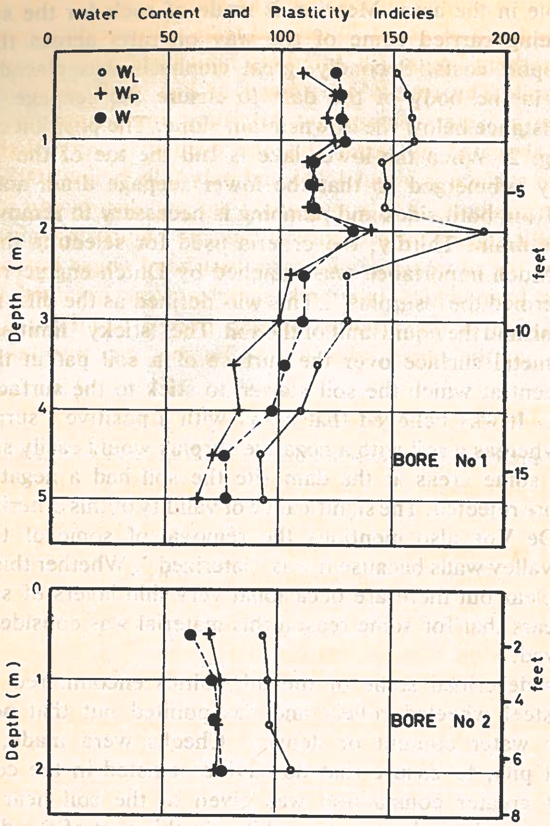

FIG. 5.—Atterberg Limits and Water Content from Bore 1 and Bore 2

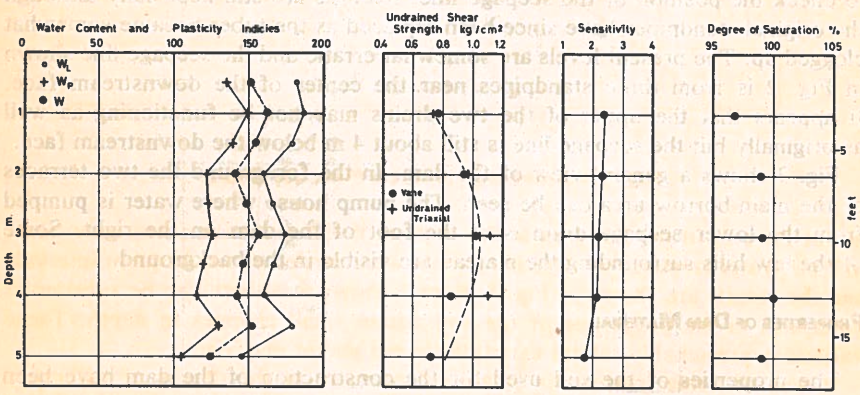

FIG. 6.—Soil Characteristics at Bore 3

taken from the two slopes labeled here as slope 1 and 2. The samples were obtained from a depth of about 70 cm and tests were made to determine water content and Atterberg limits. All Atterberg limit tests were carried out without predrying the soil. The results are shown in Fig. 4. It is seen that there is a steady decline in water content and Atterberg limits from the surface down to a depth of about 15 m. After this the decline is very slight. The water content near the surface is about 150% and only falls below 100% after a depth of 15 m. Water contents of this magnitude are not uncommon in volcanic ash soils and normally suggest the presence of the amorphous clay mineral allophane.

In addition to the aforementioned tests two hand auger boreholes were put down at points from which two bulk samples were obtained for laboratory testing The position of these bores, labeled Bore 1 and Bore 2, is shown in Fig. 1. Bore 1, at the upper edge of the borrow pit, was taken down to 5

TABLE 1.—Composition of Tjipanundjang Samples

Data (1)	Allophane (2)	Halloysite (3)	Kaolinite (4)	Gibbsite (5)	Cristobalite (6)
Bulk sample TJ 1	predominant	5–10	—	5	—
Bulk sample TJ 2	minor	predominant	trace	—	rare
Bore 1, 2m	80	15 ± 5			
Bore 1, 3.5m	40	53 ± 5			
Bore 1, 5m	25	70 ± 5			

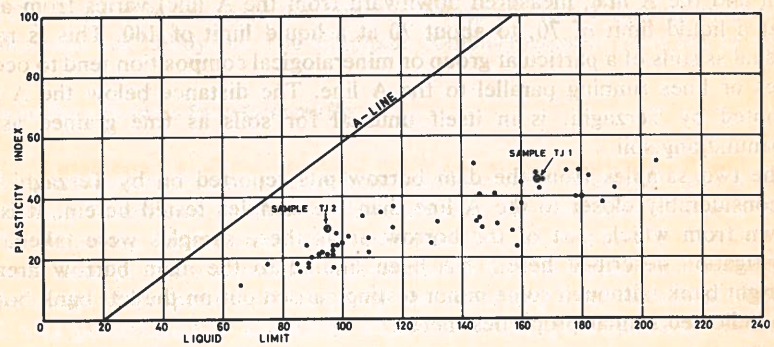

FIG. 7.—Atterberg Limits on Plasticity Chart

m but Bore 2, in the flat area at the bottom of the excavation was only taken to 2 m. Water content and Atterberg limits were carried out at frequent intervals and the results are shown in Fig. 5. Bore 1 shows that there can be substantial variations in the properties of the soil within small changes in depth. These variations presumably reflect variations in the parent material.

It was of some interest to obtain information on the degree of saturation and sensitivity of the soil and for this purpose a third hand auger hole, labeled Bore 3, was put down. From this borehole undisturbed samples were taken at 1-m intervals and measurements were made in the laboratory of undrained shear strength, sensitivity, and degree of saturation. Water contents and Atterberg limits were determined at 50-cm intervals. The results, shown in Fig. 6, show the soil to be of low sensitivity with a degree of saturation of virtually 100%. However, these sensitivity values should not be taken as being representative of the soil as a whole. The water content over the full depth of Bore 3 occupies a fairly fixed position in relation to the plastic and liquid limits (i.e., constant liquidity index) but from Fig. 4 it is seen that in some places the water content is actually above the liquid limit. When this occurs high sensitivity values can be expected as the undisturbed shear strength is fairly constant. The in situ shear strength values shown in Fig. 4 were measured at depths of 70 cm using a Pilcon type hand vane.

In neither of the boreholes taken to a depth of 5 m was water encountered, nor in the 2-m borehole at the bottom of the borrow pit excavation. There is no sign of any seepage occurring from the face of the borrow pit slope so that it appears that the ground-water level is quite deep. Despite this, however, the degree of saturation of the soil is still virtually 100%, even quite close to the surface.

In Fig. 7 all the Atterberg limit values obtained from the three boreholes and the borrow pit slopes are plotted on the plasticity chart. The figure clearly shows the very wide variation in the Atterberg limits occurring in the borrow pit material. The lowest value of the liquid limit was 66 and the highest 205. The plasticity index varies between 11 and 51. While the points show considerable scatter, their general position on the chart is typical of Indonesian andosols. They all lie well below the A line with the distance below increasing with increasing liquid limit. The average ΔI_p value (vertical distance between the point and the A line, measured downward from the A line) varies from about 20 at a liquid limit of 70, to about 70 at a liquid limit of 180. This is rather unusual as soils of a particular group or mineralogical composition tend to occupy zones or lines running parallel to the A line. The distance below the A line, as noted by Terzaghi, is in itself unusual for soils as fine grained as the Tjipanundjang soil.

The two samples from the dam borrow pits reported on by Terzaghi both lie considerably closer to the A line than the samples tested herein. It is not known from which part of the borrow areas these samples were taken. The investigation described herein has been limited to the main borrow area on the right bank, although some minor testing carried out on the left bank borrow area indicated similar properties there.

Mineralogical Composition

Mineralogical analysis has been restricted to the two bulk samples and to three samples taken from Bore 1 at depths of 2 m, 3.5 m, and 5 m. The two bulk samples have been designated TJ1 and TJ2; the former was taken from the top of the borrow pit alongside Bore 1 and the latter from the bottom of the borrow pit alongside Bore 2. The mineralogical analysis has been carried out by the Petrology Section of the New Zealand Geological Survey under the supervision of W. A. Waters, along with the analysis of a number of other samples from andosol and latosol areas in Java. A full report of this work is given elsewhere (11), and only the information on the Tjipanundjang samples

is included herein. The composition was found as shown in Table 1 (figures give percentages).

An analysis of all the results from the andosol and latosol areas shows the following three factors:

1. The predominant minerals are halloysite and allophane.
2. There is a close relationship between the natural water content and Atterberg limits on the one hand and the proportions of halloysite and allophane on the other. The natural water content and Atterberg limits rise rapidly with increasing allophane content. Natural water contents below about 90% indicate a higher proportion of halloysite than allophane, while water contents above 120% indicate a substantially higher proportion of allophane than halloysite. Water contents between these values suggest that the proportions of halloysite and allophane will not be very different.
3. In andosol areas, of which Tjipanundjang is typical, the upper layer consists predominantly of allophane but the proportion of halloysite increases with depth and becomes predominant after several areas.

Thus in Fig. 4 the upper 5 m to 10 m of soil consists predominantly of allophane but the halloysite content increases with depth and becomes predominant below 15 m. Bore 1 in Fig. 5 shows a similar trend and suggests that after about 4 m the halloysite has become predominant. Bore 2 at the bottom of the borrow pit shows that halloysite is predominant here. Bore 3 in Fig. 6 indicates that the allophane content is still predominant over the full 5 m.

In Fig. 7 the wide scatter in the values is clearly due to the varying proportions of allophane and halloysite. The allophane content increases from left to right.

Tests on Bulk Samples From Borrow Pit

As mentioned earlier two large bulk samples were obtained from a depth of between 70 cm and 1 m below the surface at the locations of Bores 1 and 2. The Atterberg Limits from these two samples, shown in Fig. 7 show that the two samples are reasonably representative of the upper and lower limit of the range of properties indicated by the Atterberg limit variations. Sample TJ 1 was predominantly allophane while TJ 2 was predominantly halloysite with minor allophane.

Particle Size, Atterberg Limit, and Specific Gravity Tests.—Tests were first carried out to determine the influence of air and oven drying on these soils. Particle size, Atterberg limits, and specific gravity determinations were made on the soil in its natural state, then after air and oven drying. Air drying here means leaving the soil spread out on a tray in the laboratory for at least 10 days. Room temperature was normally about 25° C. The results of these tests are summarized in Table 2 and the particle size curves are shown in Fig. 8. The Atterberg limit tests on the natural soil were carried out in accordance with normal procedure, except that no drying was permitted. The air-dried and oven-dried samples were sieved on the No. 40 (ASTM) sieve, mixed with water, and stored overnight in plastic bags before proceeding with the test. The particle size tests were carried out also in accordance with normal procedure using

TABLE 2.—Tests on Bulk Samples

Sample (1)	Natural water content, as a percentage (2)	Test Condition (3)	Atterberg Limits: Liquid limit (4)	Atterberg Limits: Plastic limit (5)	Atterberg Limits: Plasticity index (6)	Specific gravity (7)	Particle size: Passing number 200 (8)	Particle size: Clay fraction (9)	Standard Compaction Tests: Optiumum water content, as a percentage (10)	Standard Compaction Tests: Maximum dry density, grams per cubic centimeter (11)
TJ 1	128	Natural	165	119	46	2.80	96	65	120	0.61
		Air dried ($w = 30$)	60	53	7	2.78	37	11	55	0.96
		Oven dried	44	42	2	2.54	16	4	40	1.19
TJ 2	68	Natural	95	65	30	2.88	96	76	64	0.965
		Air dried ($w = 28$)	74	57	17	2.84	94	69	53	1.084
		Oven dried	61	46	15	2.74	81	61	44	1.145

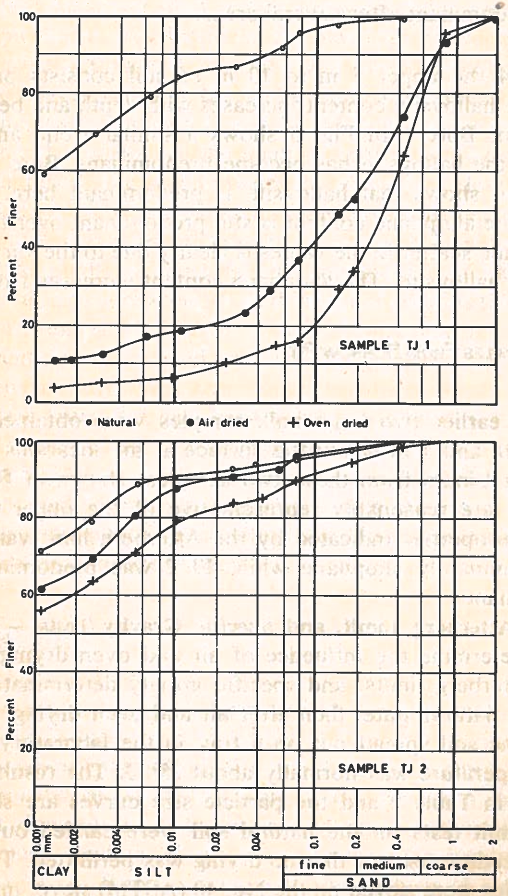

FIG. 8.—Particle Size Curves

sodium hexametaphosphate as the dispersing agent. The weight of soil used for the tests when no predrying was permitted ("natural") was determined by taking a second sample of similar size of identical material for water content determination.

It is immediately apparent that air and oven drying has a drastic effect on the properties of sample TJ 1, but that with sample TJ 2 the effect is much less pronounced, though still significant. Air or oven drying of sample TJ 1 actually changes the material from a light brown clay to a very gritty, silty sand, dark brown in color. Air or oven drying of sample TJ 2 however does not produce any readily visible effects. The material retains its cohesion and plasticity and remains yellowish brown in color. The specific gravity values given in Table 2 are apparent values only; no attempt has been made to take into account any chemical changes which may result from drying.

The particle size results are of some significance. Tested without air or oven drying gave clay fractions of 65% and 76% for samples TJ 1 and TJ 2, respectively. These values are very different from the values of 6% to 33% reported by Terzaghi. It is not clear from Terzaghi's paper whether the Sasamua and Tjipanundjang samples were tested with or without predrying. It is difficult to see how clay contents as low as 6% to 33% could be obtained from the Tjipanundjang soil unless it was dried before testing. It should be mentioned, however, that particle size measurements with andosols containing mainly allophane can be very difficult due to the extremely strong tendency of the soil to flocculate after standing for some time in the hydrometer test cylinder. With the Tjipanundjang samples this did not occur, except possibly to a minor degree. It is possible that the 65% clay fraction from TJ 1 is on the low side due to some flocculation occurrence. It was found that the dispersing agent does not appear to play a very important role in dispersing the soil. Tests were carried out by dispersing the soil with the mechanical mixer and then adding the dispersing agent to the cylinder immediately before starting sedimentation. This procedure produced identical results to those obtained by mixing the dispersing agent with the soil and allowing it to stand overnight before starting the test. The function of the dispersing agent in this case appears to be only in preventing flocculation, as the soil flocculated noticeably when the dispersing agent was omitted.

Compaction Tests.—Compaction tests using standard compactive effort were carried out on both samples. The tests were first carried out by carefully drying the soil back from its natural water content to the water content required for each point on the compaction curve. Fresh material was used each time. The tests were then repeated on the soil after air and oven drying, again using separate soil for each point. The compaction curves thus obtained are shown in Fig. 9. The drastic and irreversible effect which air or oven drying has on sample TJ 1 is clearly demonstrated by these curves. The effect with sample TJ 2 is also quite significant.

The compaction curves from sample TJ 1 are very similar to those reported

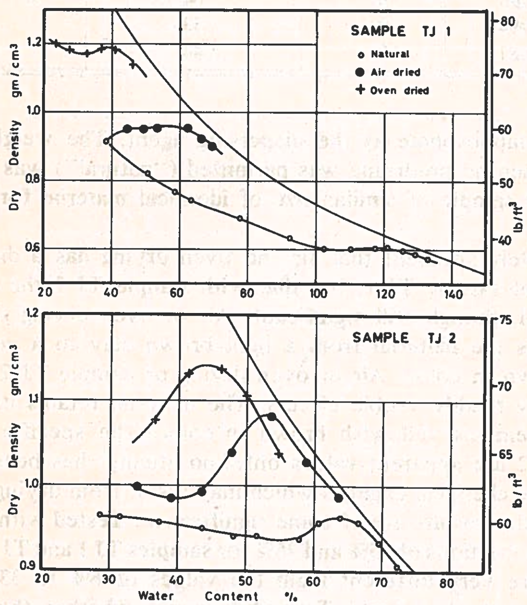

FIG. 9.—Compaction Test Results

elsewhere from tests on volcanic ash soils consisting mainly of allophane. Northey (7), for example, gives results from a New Zealand allophane soil and Frost (6) reports on results from an allophane soil in New Guinea.

The optimum water content with standard compaction is almost the same as the plastic limit, provided the treatment of the soil prior to testing is the same. The drop in plastic limit on air or oven drying is accompanied by an almost identical drop in the optimum water content. This is somewhat at variance with the results reported by Terzaghi where the optimum water content values were some 8% below the plastic limit. However, with a total sample size of only 10 lb it was probably not possible to carry out reliable compaction tests on this soil.

Shear Strength Tests.—Extensive triaxial testing has been carried out on both

samples. The samples were prepared by compacting the soil at its natural water content to the corresponding dry density from the compaction curve. Consolidated undrained tests were first carried out at cell pressures from 0.5 kg/cm^2 to 4.0 kg/cm^2 (1 kg/cm^2 = 98 kN/m^2 = 14.2 psi). Back pressure was used to saturate the samples. It was found, however, that compacting the soils at their natural water contents resulted in the expulsion of almost all air from the voids. The degree of saturation after compaction for sample TJ 1 averaged 97% and for TJ 2 averaged 98%. For the drained tests which followed back pressure saturation was therefore not considered necessary. The drained tests were carried out at cell pressures ranging from 0.5 kg/cm^2 to 5.0 kg/cm^2.

Typical curves of deviator stress and pore-pressure change versus strain from

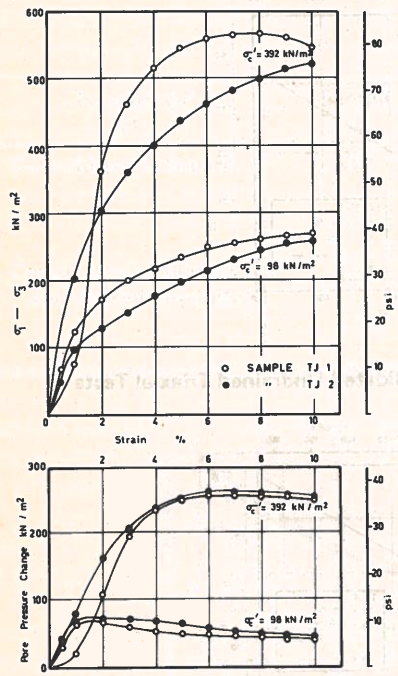

FIG. 10.—Typical Results from Consolidated Undrained Triaxial Tests

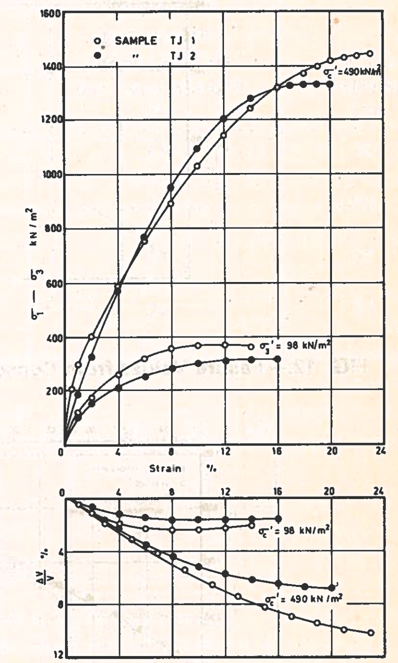

FIG. 11.—Typical Results from Drained Triaxial Tests

the consolidated undrained tests are shown in Fig. 10. The results from both samples at effective consolidation pressures (cell pressure less back pressure) of 1.0 kg/cm^2 and 4.0 kg/cm^2 are plotted together in this figure. In Fig. 11 typical results from the drained tests are shown in the form of deviator stress and volume change versus strain. Results from the two samples are again plotted together for cell pressures of 1.0 kg/cm^2 and 5.0 kg/cm^2.

It is seen from these results that the behavior of the two samples in triaxial tests is remarkably similar despite their difference in mineralogical composition and water content. The water content of TJ 1 is almost twice that of TJ 2 but this does not have an appreciable effect on the shear strength behavior

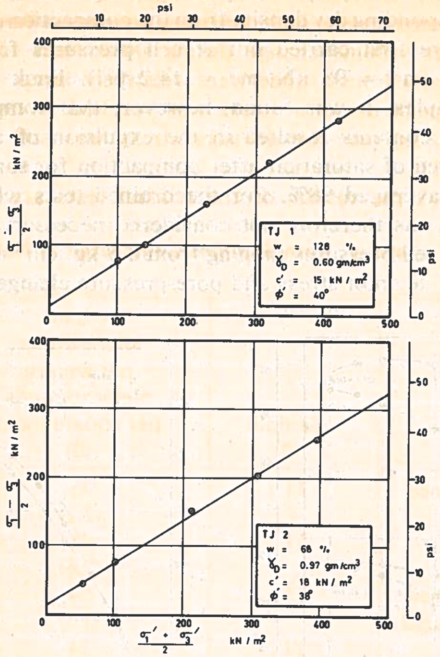

FIG. 12.—Failure Values from Consolidated Undrained Triaxial Tests

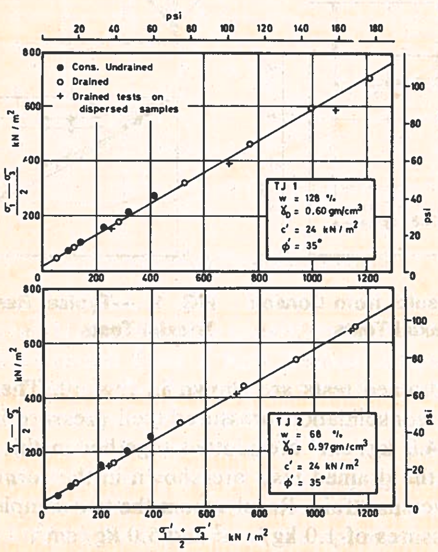

FIG. 13.—Failure Values from All Triaxial Tests

of the soil. The only difference of any magnitude is in the volume change occurring during drained tests at high confining pressures. Sample TJ 1 undergoes a substantially larger volume change during shear than TJ 2. Both samples, however, during drained tests underwent relatively large strains and volume change before the peak deviator stress was reached. With sample TJ 1 at a cell pressure of 5.0 kg/cm² the deviator stress was still rising slightly at a strain of 23% (Fig. 11).

In Figs. 12 and 13 the results of the triaxial tests are shown as plots of

TABLE 3.—Values of c' and ϕ' from Samples Compacted at Natural Water Content

Type of test (1)	Sample (2)	c', in kilonewtons per square meter (pounds per square inch) (3)	ϕ', in degrees (4)
Consolidated undrained	TJ 1	15 (2.2)	40
	TJ 2	18 (2.6)	38
Consolidated undrained and drained (averaged)	TJ 1	24 (3.5)	35
	TJ 2	24 (3.5)	35

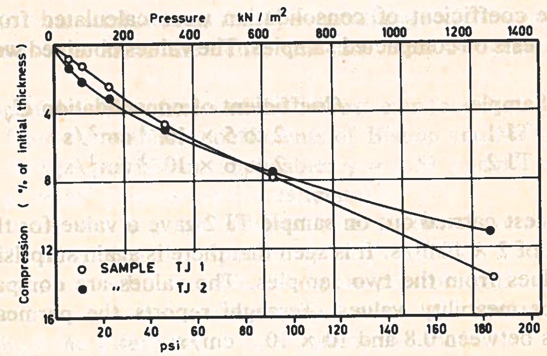

FIG. 14.—Compressibility Curves from Oedometer Tests

$(\sigma_1 - \sigma_3)/2$ versus $(\sigma_1' + \sigma_3')/2$ at failure. Fig. 12 shows the results of the consolidated undrained tests only, while Fig. 13 shows both these and the drained tests plotted together. It is seen that there is little scatter in the results, and the values obtained from the two samples are almost identical. However, the consolidated undrained tests indicated a somewhat higher ϕ' value than the drained tests. This may be due either to the different test method or to the higher stress level in the drained tests. The results are summarized in Table 3.

These ϕ' values are surprisingly high for a soil with a clay content of about 70%. Terzaghi attributes the high shear strength of Sasumua and Tjipanundjang clays to the fact that the particles are cemented together to form clusters or "spongy aggregates". However, the tests described herein do not support this view. There does not appear to be any reason for believing that the Tjipanundjang soil in its natural state consists of these spongy aggregates. In an attempt to investigate further the cluster hypothesis in this case several tests were carried out on samples which had been dispersed by mechanical stirring and then dried back. As mentioned earlier it appears that the soil is adequately dispersed by mechanical dispersion only, although this has not been proved conclusively. After dispersion in the mixer the samples were dried back to their natural water contents and compacted for triaxial testing as before. Drained tests were carried out on both samples at cell pressures of 1 kg/cm^2, 3 kg/cm^2, and 5 kg/cm^2. The results are shown in Fig. 13. It is seen that this treatment results in a slight decrease in strength with sample TJ 1, but has no effect on the strength of TJ 2. While not conclusive these tests suggest that the high shear strength is due to the nature of the individual particles and not to their existence in clusters.

Permeability and Coefficient of Consolidation.—Permeability tests were carried out on samples compacted at their natural water content in the standard compaction mould. The following values were obtained:

Sample	Coefficient of permeability k
TJ 1	3.2×10^{-8} cm/s
TJ 2	2.7×10^{-8} cm/s

Values of the coefficient of consolidation were calculated from the results of consolidation tests on compacted samples. The values obtained were as follows:

Sample	Coefficient of consolidation C_v
TJ 1	2 to 5×10^{-3} cm^2/s
TJ 2	2 to 6×10^{-3} cm^2/s

A dissipation test carried out on sample TJ 2 gave a value for the coefficient of consolidation of 2×10^{-2}/s. It is seen that there is again surprising agreement between the values from the two samples. The values are comparatively low, especially the permeability values. Terzaghi reports the permeability of the Sasumua clay as between 0.8 and 10×10^{-7} cm/s.

The preceding values of k and C_v may not give a true picture of the values actually applicable in the dam. A permeability test on an undisturbed sample from Bore 3 at a depth of 3 m gave a permeability of 4×10^{-6} cm/s, much larger than the values from the compacted soil. Beckman mentions that the dam material compacted by the rollers actually consisted of large lumps covered over by finer material which had dried out to some extent. Thus the large lumps may have stayed more or less intact with a corresponding higher permeability and coefficient of consolidation. Any drying out of the allophane soil would also have tended to result in higher permeability.

Compressibility.—The compressibility measured by consolidation tests on compacted samples at their natural water content is shown in Fig. 14. Again

the similarity in behavior of the two samples is evident. The compressibility is actually quite high, over twice that obtained from the field compressibility measurements on the Sasumua clay. The compressibility in the field with the Tjipanundjang soil may be somewhat less than that obtained from laboratory tests but the curves in Fig. 14 do suggest that the negligible settlement occurring after completion of the dam was due more to the rapid dissipation of pore pressure and the absence of secondary settlement than to the low compressibility of the material.

Stability of Dam

Detailed stability analysis of the dam has not been carried out and this section will be restricted to some general comments.

Information on pore presssures during steady seepage is available from the standpipe water level records. No direct information is available on pore pressures during construction but the settlement records mentioned by Beckman suggest that dissipation took place as the dam rose and that there were no significant pore pressures remaining at completion of the dam. Beckman states that accurate level records taken at three monthly intervals for over 1 yr after completion showed no measurable settlement. A further observation recorded by Beckman is of significance when considering the case of rapid drawdown. Beckman states that the seepage line as indicated by the water levels in the standpipes rises rapidly during periods of rainfall, and that after the rain ceases the levels return to their steady state within a few days. It would appear from this that the fall in pore pressure during drawdown may be quite rapid. Drawdown takes place during the dry season over a period of several months so that with a relatively "rigid" material such as the Tjipanundjang soil this may not constitute "rapid drawdown" at all in the sense that this term is normally used in soil mechanics.

An indication of the safety factor of the downstream face during steady seepage can be gained from the stability coefficients of Bishop and Morgenstern (1960). The following soil properties are applicable: $\gamma = 1.53$ g/cm^3; $c' = 24$ kN/m^2 (3.5 psi); and $\phi' = 35°$ (a conservative estimate).

The density value of 1.53 g/cm^3 is based on an assumed average water content for the dam fill of 85% and a specific gravity value of 2.80. With a height of 34 m this gives $c'/(\gamma H) = 0.046$. The pore-pressure ratio, r_u, on a typical slip surface will have a maximum average value of about 0.5. For the 1:3 downstream slope the safety factor thus obtained is about 1.7.

For the upstream slope, if no pore pressure is assumed during drawdown the safety factor is about 2.5. If drawdown results in an r_u value of 0.5 the safety factor falls to 1.44. It is apparent that the safety factors are the type that would be used if modern stability analysis methods had been used in the design.

The stability of the borrow area slopes is worth noting in passing. The main slope (slope 1 in Fig. 4) is just over 21 ms in height and has a slope of almost exactly 45°. It has remained intact during the 40 yr since its formation and provides a good example of the remarkable stability of these yellowish brown andosols. The undrained shear strength values shown in Figs. 4 and 6 indicate an average value of about 70kN/m^2. This gives a $c_u/(\gamma H)$ value of 0.22 and an end of construction safety factor based on total stress analysis ($\phi = 0$) of about 1.25.

Some Further Comments

It has been shown that the soil from the Tjipanundjang Dam borrow pit consists of soil made up principally of halloysite and allophane. The upper layers are predominantly allophane and those deeper down are mainly halloysite. No samples from the dam itself have been tested so that it has not been proved conclusively that a large part of the dam material consists of allophane soil. However it does not appear possible to draw any other conclusion from the evidence described previously. There is, in fact, some reason to believe that the Dutch engineers selection criteria tended to favor allophane soil rather than halloysite soil. As mentioned earlier the criteria was that the soil should have a positive "surplus." The Dutch engineers considered the Tjipanundjang soil abnormal because it was very fine grained and yet had a positive surplus. Positive surplus values are normally obtained only from silts or sandy clays of low plasticity. Clays of high plasticity give a negative surplus. However, the writer has had "surplus" determinations made on a number of samples from the borrow pit and large negative surplus values were obtained. This was without predrying the soil. Positive values can however be obtained by predrying the soil, at least with those samples which undergo substantial irreversible changes after air drying. Dutch practice at the Bandung laboratory was to dry the soil before consistency limit testing so that with soils containing a substantial amount of allophane positive values would be obtained.

The "surplus" criteria generally does not appear to be at all valid with these tropical residual soils. The red clays, being very sticky materials, have a large negative surplus value and yet are remarkably good as embankment materials.

The tests on the two bulk samples, representing the upper and lower limits of the range of Atterberg limits and natural water contents, show that neither of these properties is useful as an indication of the probable engineering properties of the soil. The wide variation in the Atterberg limits and water content is not accompanied by a variation in shear strength, compressibility, or permeability. It seems that a large part of water in sample TJ 1 does not play any part in influencing the mechanical behavior of the soil. The Atterberg limit values are, of course, still useful in identifying the soil type by the unusual position they occupy on the plasticity chart.

Perhaps the most significant fact brought out by the study of the two bulk samples is that the engineering properties are not significantly influenced by the change in mineralogical composition from allophane to halloysite (which accounts for the large variation in water content and Atterberg limits). The amorphous allophane behaves in much the same way as the crystalline halloysite and has almost identical shear strength. It appears from this that particle size and shape have even less influence on the properties of these soils than they do on normal sedimentary clays. The nature of the individual particles (or gel fragments of the allophane) and the interparticle forces appear much more important than particle size or shape.

The mineralogical composition does influence greatly the behavior of the soil when air or oven dried. The allophane soil undergoes a drastic irreversible

change when air or oven dried converting it from a clay to a slightly silty sand. The "sand" is very gritty and the particles are remarkably hard. For an explanation of the process by which the allophane forms into hard sand sized particles the reader is referred to the work of Fieldes (4). Note that it is the random allophane fragments themselves which link together to form the hard particles, and the process is not one in which some cementing agent effectively cements together already existing particles. Whether the gel-like allophane can be considered to consist of discreet particles appears debatable. However, after air or oven drying the material becomes definitely "particulate" although still not possessing crystalline structure. The nature and structure of the allophane soil is extremely complex, as explained by Fieldes (4) and Fieldes and Furkert (5).

The halloysite soil undergoes a change upon air and oven drying but not nearly as drastic as the allophane soil. The reason for this change has not been established although in this case it is probably due to the presence of minor amounts of allophane.

Conclusions

It has been shown that the soil of which the Tjipanundjang Dam is constructed is a remarkably good engineering material belonging to a group of yellowish brown clays known as andosols. The predominant minerals in this group are halloysite and allophane.

The investigation of the properties of the soil emphasises two factors which need to be taken into account when dealing with tropical residual soils. These are:

1. Test methods satisfactory for sedimentary soils involving predrying of the soil may give quite meaningless results when used for tropical soils, due to the drastic change which can take place in these soils on air or oven drying.
2. Empirical relationships based on the characteristics of sedimentary clays may not be applicable to tropical soils. In particular, the general trend toward lower shear strength with decreasing particle size is clearly not applicable to this group of soils. These soils have a high shear strength despite a high clay content. Also the trend toward lower shear strength with increasing water content and void ratio is not applicable.

No simple explanation can be put forward to account for the good engineering properties of this group of allophane and halloysite soils. The cluster hypothesis put forward by Terzaghi (8) does not appear to be a satisfactory basis for explaining the properties of the Tjipanundjang soil (and other halloysite soils) as there is no firm evidence to suggest that cemented clusters exist in the soil in its natural state. In the writer's view Terzaghi failed to take due account of the preceding two factors when putting forward the hypothesis. Experience with sedimentary clays suggested that high shear strength was not compatible with very small particle size, and the idea of clusters was supported by particle size tests carried out on soil which had been predried.

Terzaghi's data on the Tjipanundjang Dam do not appear to have been very reliable. The yellowish brown soil is decribed as red, and in the mineralogical analysis no mention is made of allophane although allophane has been shown to be a major constituent of the soil and is almost always present to some extent in the andosol soil group to which the Tjipanundjang soil belongs.

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Seismic Stability of Upper San Leandro Dam[a]

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Synopsis

The 50-yr-old hydraulic-fill Upper San Leandro Dam was analyzed for potential behavior during earthquakes using recently developed stability analysis techniques based upon postulated ground motion patterns, dynamic as well as static soil properties, and saturation patterns. The study was initiated by the owner, the East Bay Municipal Utility District, because of growing concern over safety considerations for such dams in seismically active areas which have also become highly congested metropolitan centers. The Division of Safety of Dams, State of California, subsequently directed a review of the seismic stability of the dam using these new techniques. The near catastrophe of the Lower San Fernando Dam in the San Fernando earthquake of February, 1971, certainly highlighted the vulnerability and the consequences of failure of such structures.

The study was conducted by Woodward-Lundgren and Associates, in cooperation with the staff of the District, utilizing dynamic testing techniques and analytical procedures developed at the University of California, at Berkeley, and applied to the investigation of the near collapse of Lower San Fernando Dam. H. B. Seed served as program consultant to the District for the current program. The dynamic studies reported herein were conducted under the direction of I. M. Idriss. In order to analyze the full spectrum of possible embankment behavior, the study considered several reservoir water levels, a range of material properties, and three postulated earthquake motions originating on two major faults.

The significant results of the investigation are summarized in Figs. 1 and

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