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Lessons Learned from the Analysis of a Wet Core Dam

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Lessons Learned From The Analysis Of A Wet Core Dam

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ABSTRACT

Wadaslintang Dam, constructed for the Government of Indonesia's Directorate of Irrigation, was completed in 1987 and stands approximately 120 m (394 ft) high at the maximum section with a crest length of about 700 m (2,296 ft). At the time of its completion it was the largest dam ever constructed using the wet core method of construction. Consequently, the project data constitutes a valuable case history of a large dam constructed with a soil core when it was impossible to achieve conventionally accepted standards for placement and compaction of the core zone material. The instrumentation data obtained during construction provides invaluable information on such aspects of behavior as stress distribution in the embankment, pore pressure generation and dissipation, and deformations during construction. Analysis of the data yielded insight and information on a number of aspects relevant to the analysis and design of earth structures, as well as the instrumentation of such structures, and this paper highlights several of the most significant lessons learned from this project. In addition, the project data will contribute to the database of geotechnical information on earth structures constructed in tropical regions of the world.

KEYWORDS

Dams, earth; Deformation; Earth pressure; Halloysite; Instrumentation; Pore water pressure

INTRODUCTION

Wadaslintang Dam was constructed for the Government of Indonesia's Ministry of Public Works, Directorate of Irrigation, as part of the water resource development plan for the South Kedu region of Central Java. The dam is located on the south side of the island, about 70 kilometers northwest of Yogyakarta. Project construction began in late 1982 and the embankment was completed in mid 1987. Design services and construction supervision were provided by Engineering Consultants, Inc. (ECI) of Englewood, Colorado. The dam, founded entirely upon rock, is a zoned earth and rockfill structure with internal transition, filter and drainage zones. Its height is approximately 120 m at the maximum section with a crest length of about 700 m at elevation 191 m above mean sea level. Figure 1 illustrates the general embankment geometry.

The dam site is approximately seven degrees below the equator, and the rainy season generally lasts from October to April with the site receiving approximately 3000 mm of rainfall annually. The warm and humid climate, coupled with the presence of volcanic rocks, constitute the ideal environment for the formation of the allophane and halloysite clay minerals. The core zone of the embankment was constructed of residual soils with an average 74 percent passing the No. 200 sieve (0.074 mm). X-ray diffraction analyses on selected samples revealed that essentially 100 percent of the clay fraction consisted of the clay mineral halloysite, with approximately 80 percent in its hydrated form and the remaining 20 percent in the dehydrated form (metahalloysite). Consequently, care was exercised during the preparation of samples for soil testing to minimize the potential for changes in the clay soil structure due to excessive drying. The core material had an average liquid limit of approximately 71 with an average plasticity index of approximately 30.

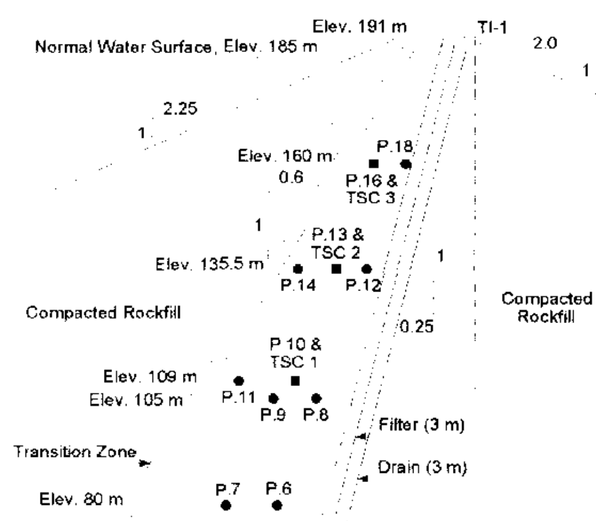

Fig. 1 Embankment geometry and instrumentation

The abundant rainfall, combined with terrace farming, resulted in natural moisture contents for the proposed core material of about 60 percent, which was about 10 percent above the Standard Proctor optimum value. These conditions made it impractical to construct the core zone to conventional standards of compaction; therefore, it was decided to build the dam using the wet core method of construction. The method was first used in the early 1950's in northern Sweden (Nilsson et al, 1955), and at the time Wadaslintang was constructed the most recent application of the method was in the dams at Monasavu Falls in Fiji (Knight et al, 1982). The core zone at Wadaslintang was constructed with soil maintained at a relatively uniform moisture content of 10 percent above the Standard Proctor optimum, and lifts of approximately 10 cm were achieved using the equivalent of Caterpillar D-6 bulldozers equipped with low ground pressure tracks. The core material could not support the weight of dump trucks or compactors; consequently, the bulldozers provided the only mechanical compaction that the material received. The in place total density of the core material averaged 1.65 Mg/m^3 (total unit weight of 103 pcf) at an average degree of saturation of 95.5 percent.

The dam embankment and foundation were instrumented with 38 pneumatic piezometers, 24 total stress cells, 13 telescoping inclinometers with annular plate magnets to monitor settlements, 6 hydraulic settlement cells, 4 installations of double fluid settlement devices, 16 surface monuments, 2 strong motion accelerometers, and 4 seepage measuring weirs. Figure 1 illustrates the distribution of piezometers (P.x) and total stress cells (TSC x) at the maximum cross section.

The data obtained from the instruments constitutes a significant contribution to the information available on the behavior of a large embankment dam constructed with a wet core, and this data along with additional observations on the behavior of the core zone is presented in detail elsewhere (Kerkes, 1990). The following paragraphs highlight a few of the more significant observations and lessons learned from the construction of this dam.

OBSERVATIONS

Among the factors influencing the pore pressures generated during construction are the stresses that develop in the embankment and the material's initial degree of saturation. The most significant factor by far is the stress state. Intuitively we know that, owing to the geometry of the embankment, the stresses that develop at any point in the dam are less than the weight of the column of material directly above that point, which would correspond to a geostatic stress state (Lambe and Whitman, 1969) or one-dimensional conditions. Nevertheless, simple one-dimensional calculations have frequently been used to estimate pore pressures on the basis of the total vertical stress increase, and it is suggested here that the observed difference between such estimates and actual measured pore pressures at other dam sites has led some engineers to conclude that (1) pore pressures were dissipating more rapidly than anticipated, and/or (2) a redistribution of stress was taking place between the core zone and supporting shell zones raising a concern over the potential for subsequent hydraulic fracturing.

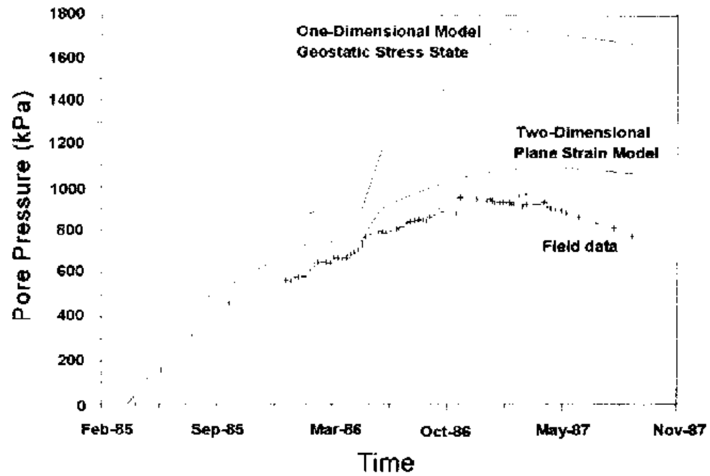

Fig. 2 Piezometer P.6

Actual occurrences of the latter are used to support the second conclusion. While there is no dispute regarding the occurrence of arching and hydraulic fracturing in zoned earth dams, it is simply suggested here that these phenomena are not always the correct explanations when field data is found to conflict with the results of analyses. It may be more appropriate to examine the validity of the assumptions used in the analyses. This point is illustrated in Fig. 2, which compares actual pore pressures measured near the base of the core zone at Wadaslintang (Fig. 1, piezometer P.6) to pore pressures estimated on the basis of (1) a one-dimensional model, which assumes geostatic stress conditions, and (2) a two-dimensional model using a simple elastic stress distribution for plane strain conditions (Kerkes, 1993). The comparison clearly shows the error that would result from the use of a one-dimensional model to predict pore pressure in the core zone. A much better estimate of pore pressures is obtained from the simple elastic model that attempts to address the two-dimensional nature of the problem, despite the fact that the model does not properly account for all the boundary conditions and the rather significant variation in material properties between the compacted rockfill shells and relatively uncompacted core material. Nevertheless, one-dimensional (geostatic) conditions will exist for some period during construction, and it would be helpful to have some idea as to when this assumption is valid.

Due to the embankment geometry, many points in the core zone will initially be very far from the upstream and downstream slopes. Consequently, one would expect that the observed total vertical stress at a point in the embankment fill would initially equal the geostatic vertical stress, which is what the field data confirms. For the same reason, however, one would also expect that the actual total vertical stresses would be less than the geostatic vertical stress by the time the embankment reached its final height, which is also confirmed by the field data. But at what point do two-dimensional conditions begin to affect the stresses and pore pressures in the embankment? Data from the total stress cells at Wadaslintang Dam tend to indicate that the vertical stress at a point in the embankment (which will be referred to here as Point A) starts to deviate from the geostatic value when the distance from the nearest point on the (upstream or downstream) slope and a vertical line through Point A is less than about two times the height of material directly over Point A.

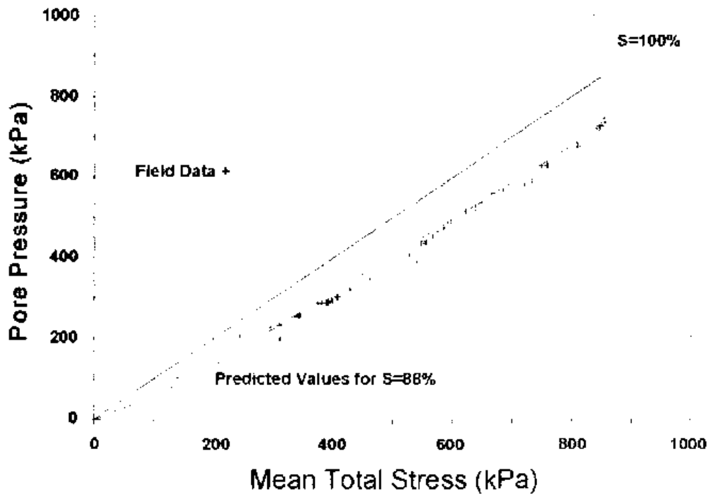

Fig. 3 Total stress cell TSC 1 and piezometer P.10

It is important to note, however, when comparing measured pore pressures to estimated stresses at a piezometer location that the observed pore pressures will be less than the estiamted stresses when the degree of saturation is less than 100 percent. Despite the relatively high average degree of saturation of the core material used at Wadaslintang, the effect of the soil's initial partial degree of saturation was clearly observed and is illustrated in Fig. 3, which compares the actual observed pore pressures to the mean total stress change at the center of the core zone (Fig. 1, piezometer P.10 and the three total stress cells at TSC 1). Superimposed over the data is a curve representing the pore pressure response for a soil with an initial degree of saturation of 88.1 percent (two standard deviations less than the average value at Wadaslintang). This curve was developed using the concepts originally proposed by Hilf (Hilf, 1948), but modified slightly to use mean total stress rather than total vertical stress. Soils compacted to conventional standards will have a degree of saturation in the low to mid 80 percent range; consequently, an even greater effect on the observed pore pressures should be expected for such conditions.

The irregular shape of the curves for the estimated pore pressures in Fig. 2 reflect the actual construction sequence followed at Wadaslintang, and the near horizontal segments of those curves for early 1986 are due to construction delays that occurred at that time. It is relevant to note, however, that pore pressures continued to increase over that period, particularly in piezometers P.6 and P.7. This phenomenon has been observed by others and has been attributed to creep behavior in the clay (Leroueil, 1994).

Regarding attempts to monitor settlements in the core zone during construction, essentially no data of value was obtained from the hydraulic settlement cells in the core zone prior to their failure, and several attempts to obtain settlement data using telescoping inclinometers equipped with annular plate magnets were unsuccessful due to excessive distortion of the inclinometer casings. The only reliable settlement data available is that obtained from the double fluid settlement devices (DFSD), and even that data does not provide a complete picture of the deformations that occurred in the core zone. No data is available to describe the settlement of the core during construction prior to installation of the respective DFSD loops, and the deformations in the core even caused some of the DFSD loops to fail. Nevertheless, using the available data and considering the volumetric strain associated with compression of the air voids in the partially saturated core material, it is possible to account for settlements in excess of 4 m at the maximum section occurring during construction.

When the dam had reached a height of approximately 83 m at the maximum section, measurements made in inclinometer installation TI-1, located downstream of the core zone and shown in Fig. 1, indicated that movement was occurring in the upstream direction above elevation 112 m, which was approximately one third the height of the embankment. The total observed movement was less than 10 cm and occurred over a period of several months; however, it raised some concerns at the time since any horizontal movements in the shell zones were expected to be away from the center of the embankment, as generally observed in other rockfill dams (Wilson, 1975, and Penman, 1986). Similar movements were not observed in any of the other inclinometers; however, there were only two other inclinometers similarly located at embankment sections that were not as high. This is one observation and as such may be an anomaly; however, it is noted here in the event that similar observations are ever made at other dams. In the course of studying this occurrence, the following was noted. For about the first 5 to 10 m of fill placed over the total stress cells, stresses approximately equal to the vertical weight of fill over the installations were recorded on each of the three orthogonal instruments, at several of the installations, indicating that the fill was acting in the manner of a dense fluid. This is consistent with behavior observed at two dams constructed with wet cores in Sweden (Lofquist, 1951). Under this condition the core material can be assumed to possess essentially no shear strength and exert stresses which are close to isotropic at all points in the zone and at its boundaries. As construction continued an anisotropic stress state developed. Field data confirms that while both the vertical and horizontal total stresses continued to increase, the total stress ratio decreased from 1 to about 0.7. Turning attention to the shell material, the rockfill not only had a higher density (2.21 Mg/m^3, or a unit weight of 138 pcf), but it was compacted by 6 passes of a 10 ton vibratory roller. It has been established that the compaction process can result in significant increases in residual lateral earth pressures that can be many times greater than the theoretical at rest values, and may even approach passive earth pressure magnitudes (Duncan et al, 1986). Therefore, a significant difference existed between the lateral stresses in the shell and the core. However, the stresses acting in the same direction on an element of material at any point in the dam must sum to zero in order for equilibrium to exist. Consequently, it is reasonable to imagine the development of a zone in the downstream shell adjacent to the core that strains or deforms toward the core sufficiently to reduce the lateral stresses in the shell zone to a level approaching the active state as the core simultaneously mobilized passive resistance. The strains need not be large to achieve this; however, in anticipation of the potential for such behavior it would be prudent for the dam designer to provide for wide filter and drain zones when employing the wet core construction method. As a final note, the point above which the upstream movement was observed (elevation 112 m) was approximately 20 m away from the core zone, which gives some idea of the width of the affected zone.

CONCLUSIONS

The wet core method of construction has proven to be a viable approach when site conditions make it impractical to apply conventional standards of compaction to the construction of the core zone; however, the designer should anticipate significant deformations in the core along with the generation of significant pore pressures. Regarding attempts to estimate pore pressures, several important points should be kept in mind. Of all the factors influencing pore pressure generation in soil, stress state is quite probably the most significant factor and remains one of the most difficult to predict. The data obtained from the instruments at Wadaslintang clearly illustrate the error that can result if pore pressure estimates are made without considering the two-dimensional nature of the problem. Reasonable agreement was found between observed pore pressures and predicted values based on mean total stresses computed using a simple two-dimensional plane strain model based on elastic theory. A finite element analysis did not yield a better estimate of the stresses in the embankment, despite its ability to more closely model the boundary conditions and difference in material properties of the different zones. The field data also illustrates the effect of the soil's initial partial degree of saturation on the pore pressures that develop in the core zone; consequently, it is important to obtain information on the actual degree of saturation of the soil in the immediate vicinity of the piezometers for subsequent use in interpreting the field data. While the behavior of partially saturated soil is complex, application of the simple concepts originally proposed by Hilf were found to yield quite good agreement with the field data. It is not suggested here that theory be abandoned and boundary conditions ignored, but rather that simple techniques not be eliminated as a design tool simply because of their limitations. As Lambe stated in his Rankine Lecture, "the engineer should not be influenced by the sophistication of his prediction techniques" (Lambe, 1973).

When developing a instrumentation plan for an embankment, groups of total stress cells (placed in three orthogonal planes) should be installed in conjunction with piezometers. Information on pore pressures without complimentary information on total stresses yields a very limited picture of what is happening in the embankment. Furthermore, confidence in the field data is increased when independent data from the two different instruments reflect the same trends and behavior patterns. In addition, the designer should install redundant instruments in the embankment in anticipation of the inevitable failures that will result from construction accidents and/or instrument problems, particularly when significant deformations are anticipated in an earth or rockfill structure.

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