Stability considerations for Guri embankment dams

Considérations sur la stabilité des barrages en remblai de Guri

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SYNOPSIS. The foundation materials for the Guri Embankment dams showed unusual behavior regarding the compressibility, development of high pore pressure during shear and low strength parameters. This article describes the special stability problems that could result from these characteristics and measures adopted to achieve a safe design.

INTRODUCTION

The final stage of the Guri project included the construction of two major earth and rockfill dams flanking the main concrete gravity dams. These dams, termed as the left and right embankment dams have a total length of 5,500 m with a maximum height of 90 m and required a total fill volume of 57,000.000 m^3. In addition, a total of 12 marginal dikes were constructed around the reservoir rim.

The foundation materials under the embankment dams generally consist of decomposed granitic gneiss varying in thickness from a few to as much as 70 m. At certain locations quartzite bands cross the embankment alignment somewhat obliquely. The decomposed granitic gneiss, generally known as saprolite has a porous structure, low densities, low degree of saturation and high compressibility.

Some of the foundation soils showed a peculiar problem of collapse by undergoing pronounced settlements on wetting at a certain load. To understand this phenomenon numerous field and laboratory tests were performed. The problem of collapse for these soils and the remedial measures undertaken are discussed by Prusza and Choudry (1979). Also the geotechnical properties of these soils are discussed in more details by De Sola (1985), Prusza et al, (1983) and Medina and Liu (1982).

DESIGN PARAMETERS SELECTION

Isotropic and anisotropic consolidation (CIU) and (CAU) tests were performed on the block samples obtained from the trenches excavated in the foundation of the left and right embankment dams. The specimens were saturated before testing, and for the CAU tests the specimens were consolidated to a stress level under a ratio $K=\sigma_1'/\sigma_3'=0.5$. Fig. 1 and 2 show the typical stress paths of two samples tested under CIU and CAU conditions respectively.

The foundation soil shows a "bond structure" at low stress. As the stress is increased, a rapid development of pore pressure occurs, showing a behavior similar to a normally consolidation clay. This bond structure seems to be destroyed when the effective stress exceeds 300 to 500 kPa.

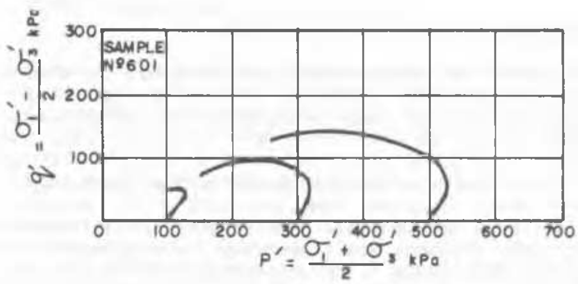

Fig. 1 - Typical stress paths of foundation material in an isotropically consolidated undrained test (CIU).

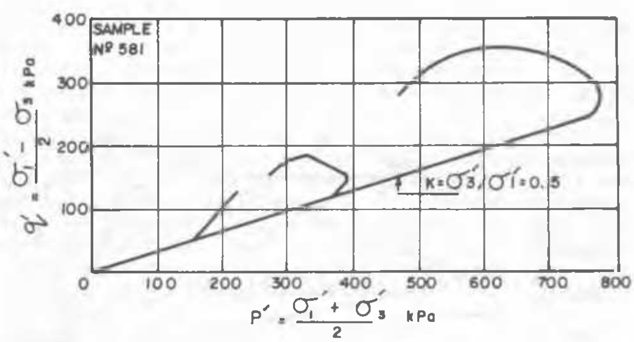

Fig. 2 - Typical stress paths of foundation material in an anisotropically consolidated undrained test (CAU).

The behavior of the foundation soil differed in some respects in CIU tests compared with CAU tests.

1.- The peak strength based on the maximum deviator stress was higher in CAU tests compared to that in CIU tests.

2.- A drop in the post peak strength, causing undrained brittleness, is observed in CAU tests, whereas in CIU tests the generation of pore pressure is not sufficient to result in a significant drop in the post peak strength.

Brittleness as defined by Vaughan (1978) is the "propensity for the strength to drop from a peak to a residual with increased strain or displacement". He also suggested that this undrained brittleness could lead to the development of a slip at reduced overall strength if the rate of loading was faster than the dissipation of locally generated excess pore pressures.

Under these conditions it was assumed that a progressive failure could occur and the operational strength would be less than the peak strength, although greater than the residual strength. Taking this into account, the parameters for the stability analysis were selected at 10% strain in CAU tests. This approach resulted in c'=21 kPa and ϕ=21.5°.

Drained shear box tests were also performed on some samples, however the residual drained strength was generally higher than that indicated by the parameters selected.

Test results for specimens compacted in the laboratory and for those obtained from the test fills showed that the compacted material was essentially dilatent whith high drained and undrained strengths and negligible brittleness. The effective friction angle based on the peak deviator stress and on the maximum obliquity was about the same. A shear strength of ϕ' = 32° with no cohesion was adopted in the design.

STABILITY ANALYSIS

In this article we have confined our discussions to the static stability analysis. The dynamic stability analysis was also performed and is described by Carrera et al (1979), elsewhere. There was considerable uncertainty in selecting appropriate failure criteria for the compressible foundation soils of Guri, due to the complexity of the failure mechanism involved and the stress path dependence of failure in such soils. A continuous program of investigation and surveillance, involving laboratory tests and instrumentation was implemented in order to reassess the design assumptions at different stages of construction and operation.

The static stability of the embankment dams was studied using a variety of computer and hand type methods of analysis. The purpose of using different methods was to determine the effects of different assumptions regarding side force's magnitude and rotation of the principal stresses on the overall computed factor of safety along a failure surface.

To simplify the stability analysis, the earthfill embankment section was assumed to be homogeneous, i.e., the relatively thin chimney, blanket drains and riprap were assumed to have the material properties of the impervious fill. This was considered to be a conservative assumption as far as the factor of safety is concerned, and helps to eliminate problems when the computer encounters thin slices of different materials. Consequently, the most important material properties are the impervious fill and the decomposed gneiss foundation material through which all of the critical failure surfaces pass.

The parameters mentioned before were used for the critical cases of steady state seepage with full reservoir level and rapid drawdown, in both cases using effective stress analysis. A safety factor of 1.6

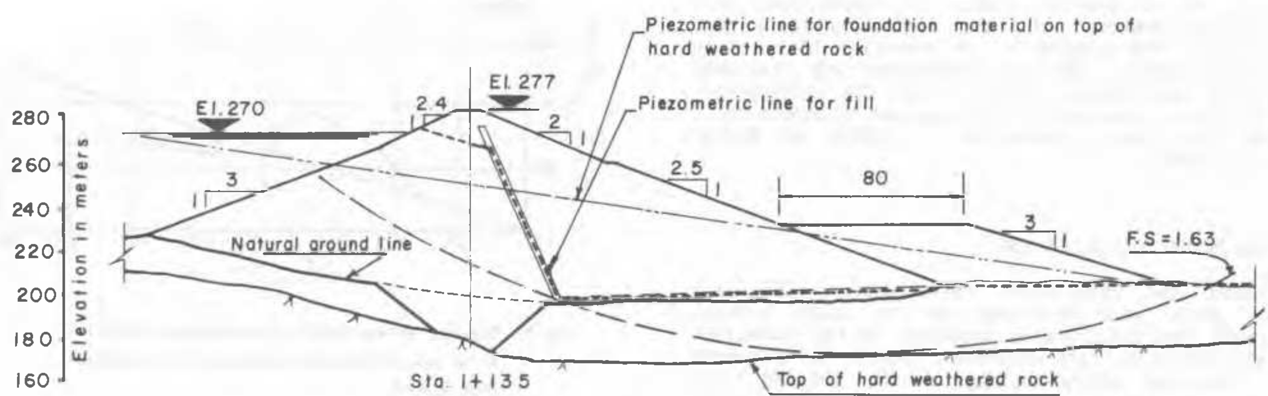

Fig. 3 - Right embankment dam, steady state seepage condition.
With narrow core trench, 80 m. wide berm is required.

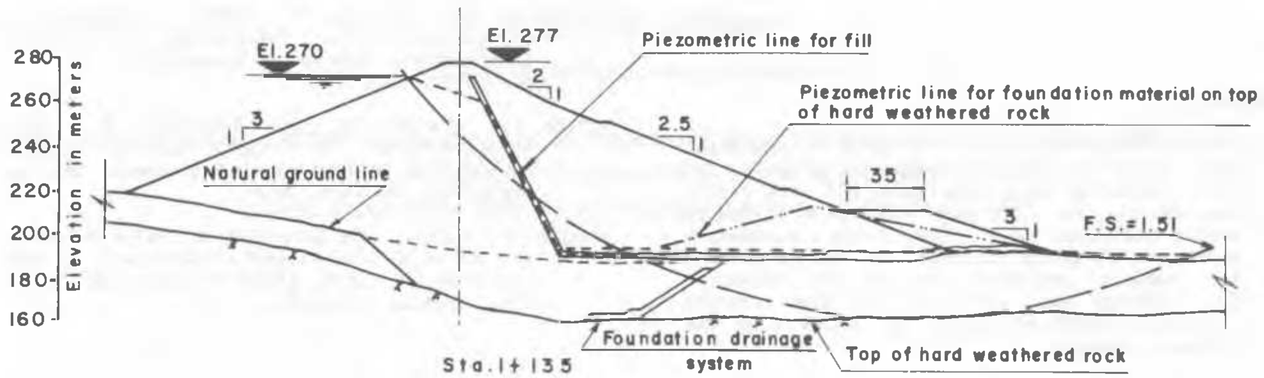

Fig. 4 - Right embankment dam, steady state seepage condition.
With wider core trench and provision of drainage system, the requirement of berm is reduced to 35m.

was obtained for the downstream slope with steady state seepage and 1.4 for the upstream slope with drawdown.

If parameters based on peak deviator stress had been used for the foundation materials, a factor of safety of 2.1 would have resulted.

The minimum factors of safety required in the design were as follows:

Steady state seepage	1.6
End of construction	1.2
Rapid drawdown	1.2

A higher factor of safety for the steady seepage condition accounted for the possibility of high pore pressure generation in the foundation during undrained shear.

A typical section used in the stability analysis is shown in Fig. 3. Under the steady seepage conditions, large weighting berms were required. These berms would spread the load of the embankment over the foundation thus reducing the possibility of differential settlements and cracking due to foundation compressibility. However, the weighting berms would not prevent the high stress concentration at the central portion of the embankment, where the behavior of the porous soil was more uncertain relative to that of the compacted fill. It was decided to excavate the porous soil to its full depth downstream of the core trench, or use a mixed approach, by excavating to full depth beneath the central portion of the embankment and adding relatively smaller berms in those portions of the dam where the thickness of the porous soil was excessive (Fig. 4).

For the left embankment dam, the porous soil was excavated to the rock surface under the whole downstream shell, thus eliminating the berm completely. Under this embankment the thickness of the porous material was relatively small.

For the right embankment, where the thickness of the porous material was higher, the mixed approach was adopted. This involved widening the core trench under the central portion of the dam, thereby reducing the size of the berm significantly. In order to control the pore pressure development an additional drainage system was used in the downstream portion of the core trench for this embankment (Fig. 4).

Once the foundation material of uncertain characteristics was removed under the downstream shell of the central portion of the dam, the factor of safety for the steady state seepage condition was reduced to 1.5.

REASSESSMENT OF DESIGN

By the end of 1984, the construction of the embankment dams was completed, and the reservoir was raised by 20 m of its total 55 m raise. Static stability analysis was performed to reassess the stability of the dams using the actual pore pressure distribution and shear strengths determined by the measurements of the instruments and the testing programs implemented during construction. Twelve sections were analysed incorporating the updated geological and topographical information. Since the average shear strength of the samples obtained during construction was very close to that assumed in the design, the strength parameters were not changed. From the instruments it was also observed that the foundation material did not develop any pore pressures during construction and the impervious compacted fill showed a pore pressure of 10% of the superimposed load.

The minimum safety factors computed were as follows:

Steady state seepage	1.56
End of construction	1.36
Rapid drawdown	1.20

which are within the original design assumptions.

These analyses were repeated at the time when reservoir reached its full level and steady state seepage conditions were established (De Fries, 1987) confirming that the safety factors were within the limits indicated above.

CONCLUSIONS

1.- The foundation materials for the Guri embankment dams showed low shear strength parameters. The tests indicated that these materials were highly compressible and developed high pore pressures during undrained shear. The strength parameters were selected at 10% strain in CAU test to account for runaway undrained failure and collapse. This strength was lower than the peak strength at maximum deviator stress but higher than the residual strength.

2.- Initial stability analysis required large weighting berms both for the left and right enbankment dams, which, in addition to giving adequate factor of safety, would spread the embankment load over the foundation reducing the differential settlements.

3.- The weighting berms would not prevent the high stress concentration at the central portion of the embankment, where the behavior of the porous soil was more uncertain relative to that of the compacted fill.

4.- For the right embankment dam a wider core trench was excavated to reduce the high stress concentration under the central portion of the dam and reducing the size of the berm.

5.- For the left embankment dam the porous material was excavated to the rock surface under the whole downstream shell, eliminating the berm completely.

6.- The reassessment of the stability after the completion of the embankment and raising of the reservoir indicated factors of safety within the limits of the original design.

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