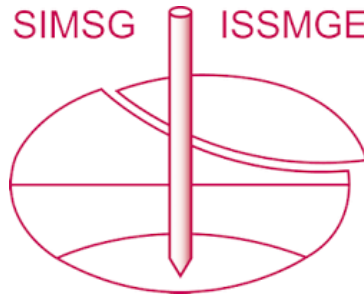


INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING



This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

<https://www.issmge.org/publications/online-library>

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.



A CASE STUDY OF ROCKFILL DAM: STABILITY EVALUATION AND REMEDIAL TREATMENT

ETUDE D'UNCAS CONCRET D'UN BARRAGE EN ENROCHEMENTS EN VRAC: EVALUATION DE STABILITE ET TRAITEMENT REPARATEUR

Hong Sung-Wan Sohn Joon-Ik Bae Gyu-Jin Ahn Sang-Ro
Um Yong-Sup Park Eun-Young

Geotechnical Engineering Division
Korea Institute of Construction Technology (KICT), Korea

SYNOPSIS: Described in this paper are the stability evaluation and remedial treatments for an earth-core rockfill dam which experienced serious open cracks on the crest. Data obtained from the site investigation, the field instrumentation and the numerical simulation have been inclusively analysed to characterize the causes of the crack development. The effect of rapid drawdown of the water level has been evaluated as a possible cause to induce the differential settlement between the dam core and adjacent zones.

1. INTRODUCTION

Comprehensive investigations have been performed to identify the potential causes of the cracks shown in the dam crest. The dam investigated for the study is one of two reservoir dams constructed for an underground pumped-storage power plant which is located in the south-eastern area of Korean Peninsula. The nominal capacity of electric power generation is 600 MW per year. The water level change of the upper reservoir for operating the power generator was about 20 m per day in a storage-and-emptying cycle.

The construction of the upper dam which was studied in this paper began in October 1979 and completed in June 1984. The impounding was completed in November 1985. The first set of cracks were observed on the dam crest in January 1987, 15 months after the beginning of operation. Two more sets of cracks occurred after the first one although they had been treated with subsequent remedial works. The crack openings were filled with the slurry grouting. Recently, cracks were detected again and considered to be of progressive nature. Further crack propagation might affect the overall stability of the dam body.

This paper discusses about the stability evaluation of the dam and the possible remedial measures based on the analyses of the data obtained from the site investigation, the field measurement and the numerical prediction.

2. GENERAL DESCRIPTION OF THE DAM AND SITE GEOLOGY

2.1 Site Geology

The upper reservoir dam is located in the narrow valley. Volcanic rock of the Cretaceous Upper Yuchun Group is distributed irregularly and widely in the dam site. The right side and

downstream area of the dam consist of the rhyolite ashly tuff of which colour is mostly gray and light greenish gray. And it includes a quartz, arkose, and a little rock-plate. The upstream area of the dam consists of Kangha tuff.

2.2 Dam Dimensions and Construction Materials

The dam is an earth core rockfill dam. The maximum height and length of the crest are 86 m and 269 m respectively. The typical cross section of the dam is shown in Fig. 1. The engineering properties of the dam construction materials are listed in Table 1. The engineering properties of the core were determined from laboratory tests and were obtained from the design reports.

Construction materials for the dam were borrowed from five distinct mountain areas near the dam site and mainly consist of residual soils and colluvial deposits. The core materials obtained by removing rubbles greater than 150 mm size are classified as well-graded cohesive soils. The filter materials were from the river-bed deposits. The inner filter layers consist of uniform fine sands mixed with clayey soils. The outer filter layers consisted of the coarse river sands. For the shell materials, the mucks obtained during the excavation of the diversion tunnel and power house were used. The remaining materials were borrowed from the adjacent quarry site.

3. ON-SITE INVESTIGATION

Fig. 2. shows the typical crack pattern. The longitudinal cracks occurred four times, in January 1987, in February 1989, in February 1991, and in August 1992. The length of the cracks ranged from 21 to 130 m and the width of the cracks ranged from 0.4 to 5.0 cm. A 6 m-long transverse cracks were also appeared. After

first longitudinal crack was appeared, the non-pressure groutings

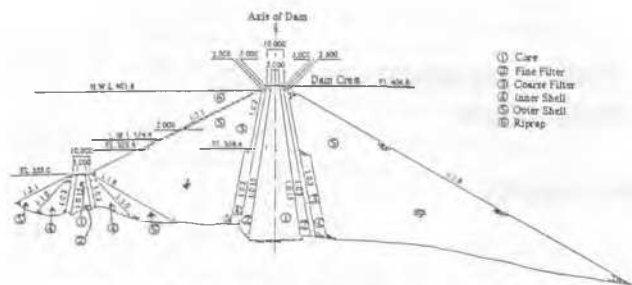


Fig. 1. Typical Cross Section of the Upper Dam

Table 1. Engineering Properties of the Upper Dam Zones

Materials Properties	Core	Fine filter	Coarse filter	Inner shell	Outer shell
Specific gravity	2.67	2.63	2.63	2.63	2.63
Water content(%)	17.3	5	5	3	3
Unit weight(t/m ³)					
dry (γ_d)	1.723	1.9	1.9	1.8	1.8
Saturated(γ_{sat})	2.077	2.1	2.1	2.1	2.1
Coefficient of permeability(cm/sec)	3.3×10^{-6}	1×10^{-4}	1×10^{-3}	1×10^{-2}	Free draining
Cohesion(t/m ²)(c')	3.1	0.0	0.0	0.0	0.0
Angle of internal friction(ϕ')(deg)	29	33	33	36	40

were used for the remedial treatments. Grouting materials used were the bentonite mixed with cement and water.

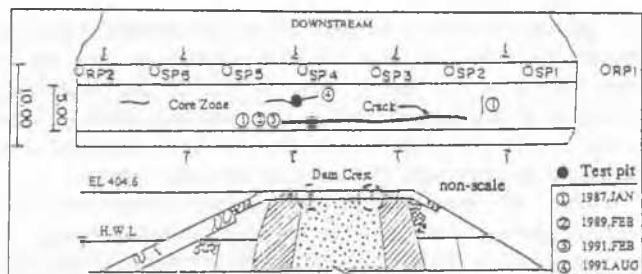


Fig. 2. Typical Cracks on the crest of the Upper Dam

Test pit was made in order to observe the depth and direction of crack development. The position of the test pit was at SP No.4., 180 m from the right abutment. Before the test pit excavation, non-soluble white paint was poured into the crack opening to trace the crack development inside the dam core.

It was observed that the cracks developed from the crest surface and continued vertically downward and laterally in the longitudinal direction, but they disappeared deep in the dam core. Down to the excavation depth of 1.4 m the crack surface coloured with the white paint was clearly visible and the width of crack opening was measured approximately 5 cm. From this depth the trace of white paint got less clear with the width of crack opening getting reduced. Also the trace of bentonite fill was shown. The bentonite was poured into the cracks in the previous repair works.

The total excavation was made down to 1.9 m where no sign of

crack development was observed. At the depth of 1.8 m the trace of white paint and the bentonite were rarely seen. But there should exist some weakening zone below this level which could not be identified from the field observation. The weakening zone characterized by means of numerical modelling will be discussed next.

The vertical direction of the crack surface indicates that the crack development would attribute more likely to the differential settlement taken place in a local zone than to the global slope instability condition of shear sliding. The considered possible causes of the differential settlements are the effective stress change induced by the repeated rapid drawdown of the reservoir water level, the possibility of the particle migration of the fine core material due to the high hydraulic gradient or poor condition of filter materials and improper condition of compacted clay core.

4. FIELD MEASUREMENT AND NUMERICAL PREDICTION

4.1 Analyses of the Field Measurement Data

Field data include the surface settlement, horizontal displacement, and pore water pressure. Surface settlement points are located on the crest and the slopes of the dam as shown in Fig. 3.

As recorded in October 1992, the settlements at the dam crest ranged from 3.3 cm to 81.5 cm. Fig. 3. Time-dependent settlements of the dam crest ranged from 3.3 cm to 81.5 cm. The settlement is largest at the point USP4. According to the empirical formula ($S=0.001 H^{3/2}$; where H is the dam height) suggested by the Lawton and Lester(1964), the final settlement is calculated to be 79.8 cm which is similar to the actual settlement measured.

When impounding began, settlements increased sharply and continued smoothly after the impounding was completed. Data for horizontal displacements show small horizontal displacement toward the downstream side in general.

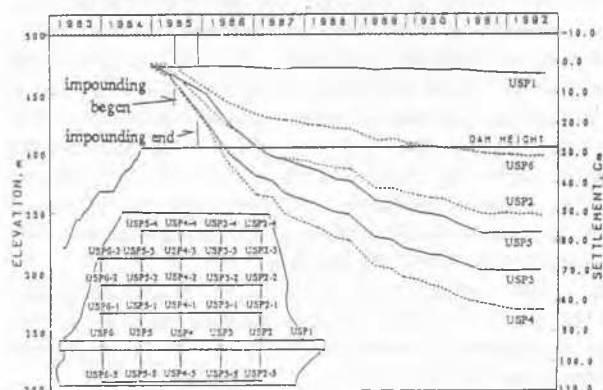


Fig. 3. Time-dependent settlements of the dam

Number of pore water pressure gages were installed in the core and adjacent filter zone as shown in Fig. 4. As shown in the figure, pore water pressure were increased rapidly in the early construction stage and then remained nearly constant in the latter construction stage. When impounding began, they were increased rapidly, and then started to slowly converge at smaller values. In

case of the UFP1 and UP1, pore water pressure were decreased a little from the early 1986, showed a random feature between 1987 and 1988, and increased from the early 1989.

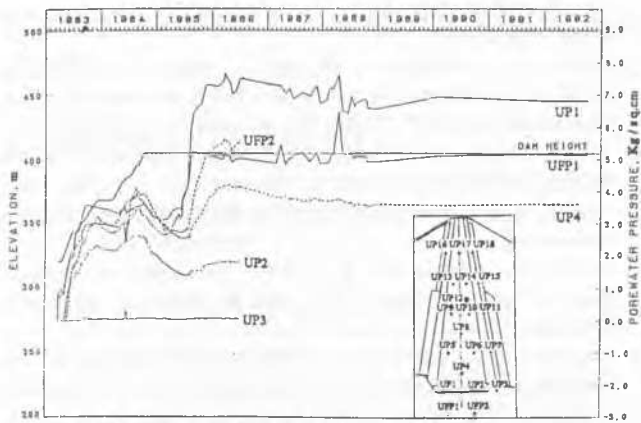


Fig. 4. Time-dependent variations of the pore pressure

4.2 Numerical Analyses

A series of FEM analyses were performed to simulate the dam behaviour from the construction stage up to the present. The cross section for the analysis was selected at sta. 1+20 point where field data were available. Two dimensional finite element modelling was carried out based on the incremental method and zoning. The incremental construction of embankment was simulated as well as the impounding and daily drawdown of the water level. Embankments were divided into five zones as shown in Fig. 1. Input data for the Cam-clay model were determined based on the laboratory test data.

Fig. 5 shows the deformation pattern in the dam structure, which indicates a large settlement in the core and in the upstream shell portion. As a consequence, the crest is tilted toward the upstream direction. At the end of construction stage, the maximum settlement is 1.2 m near the center of the embankment. It continuously increased to 2.1 m in the upstream filter zone.

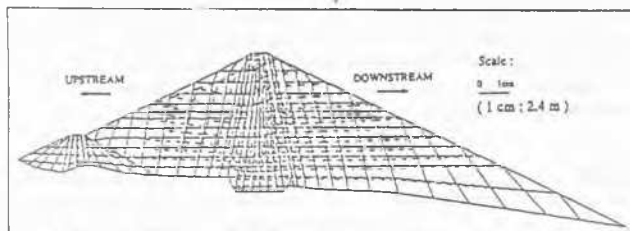


Fig. 5. Pattern of deformations in the dam structure

Fig. 6 shows the major and minor principal stress vectors calculated for the current dam condition. It indicates that the elements of the downstream side show greater magnitude of stress levels than those of upstream side. This is because the pore water pressure is higher in the upstream side. And the rotation of principal stress vectors are remarkable along the interface between the core and the filter zones. This attributes to the development of shear stresses induced due to the differential

settlement between the core and adjacent zones.

In order to evaluate the stability condition the stress ratio F_r was calculated for each element as follows:

$$F_r = \frac{(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_f}$$

where $(\sigma_1 - \sigma_3)$: the deviator stress at the current state
 $(\sigma_1 - \sigma_3)_f$: the deviator stress at the failure state

The calculated stress ratios range in general approximately from 0.3 to 0.65 except the regions close to "A" and "B" shown in Fig. 6. In these regions, F_r was calculated to be close to one. The possible cause of the higher value of stress ratio in the zone "A" is considered to be a numerical difficulty. The region "A" is the surrounding of the boundary between the part which was used as a coffer dam and the main body. The region "B" is the area from the dam crest to the depth of approximately 15 m. The higher value of stress ratio in this zone is considered to be due to the excessive differential settlement developed between the core and filter zones. Region "A" is also the area where the largest differential settlement is taken place as shown in Fig. 5. The field investigation data indicated that the crack depth was only 1.8 m deep, but the weakening zone to have been formed below the cracks could be much deeper as this numerical data indicates. For a preciser analysis finer mesh generation is required.

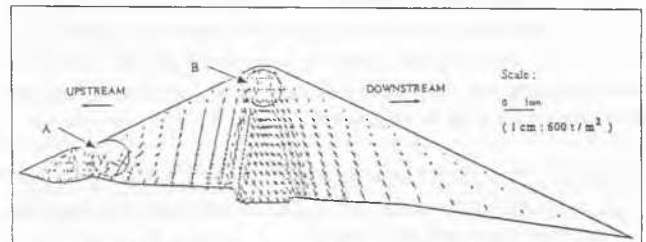


Fig. 6. Principal stress vectors at the present

5. Stability Evaluation and Remedial Treatment

As discussed before the major cause of longitudinal cracks contemplated so far is the differential settlement developed between the core and adjacent zones. Common causes of this type of crack occurring in the center-core rockfill dam attribute to the nonuniformity in stress level of overburden, material or compaction condition and foundation condition, etc. But in this case another important factor to be considered is the rapid drawdown condition of the reservoir water level.

The depth of longitudinal crack has been investigated to be approximately 2 m. Below the cracks there exists some weakening zone of which depth is numerically predicted to be about 10 to 13 m. These longitudinal cracks are judged to be hardly any imminent threats to the dam safety because they do not create direct leakage channels and would be self-healed in the deeper zone due to the lateral confinement. But in order to cope with any possible long term or short term danger remedial treatment has been suggested to solve the crack problem.

The remedial treatment employed up to now was the injection and

filling of the bentonite solution into the crack openings but it could not be an fundamental solution. The recommendations are made tentatively as shown in Table 2. Installation of instruments was been also recommended to further include crack gages and inclinometers. The final selection of the fundamental remedial works and detailed plans must be set up after the full analyses of all available informations.

Table 2. Tentative Remedial treatments

Remedial plans		Merits	Demerits	Effect
Grouting	With pressure	- Simple work - Economical	- Possibility of hydraulic fracture - Hard pressure control	- Short term
	Without pressure	- Simple work - Economical	- Poor contact	-
Replacement with ground reinforcement		- Economical when crack depth is not deep	- Technical problem when crack is deep and large - Uneconomical	- Long term

6. CONCLUSIONS

A case study has been performed in order to evaluate the stability of a clay-core rockfill dam which experienced serious cracks on the crest. The behaviour of the dam has been studied based on the data obtained from the site investigation, the field instrumentation and the numerical simulation. Following are the conclusions made from the analyses:

- 1) Most of the cracks were developed in the longitudinal direction and attribute to the differential settlements between the clay core and the filter zone.
- 2) The major cause of the differential settlement including cracks has been considered to be the uneven distribution of additional effective stresses formed in the dam body by the rapid drawdown of the reservoir water level.
- 3) The numerical prediction data indicates that the damaged part of the embankment would be from the crest to be depth of approximately 15 m in the clay core including the crack zone plus the weakening zone below.
- 4) Remedial plans have been tentatively recommended to solve the crack problem : grouting application into the existing cracks and the soil replacement with ground reinforcement for the short term and long term treatments respectively.

REFERENCES

1. Borja I. Ronald(1992), "Movement of Slopes during Rapid and Slow Drawdown", Stability and Performance of Slopes and Embankments-II, Vol.1, Geotechnical Special Publication No.31, ASCE, pp.403-413.
2. Clough R.W. and Woodward III(1967), "Analysis of Embankment Stresses and Deformations", Journal of Soil Mechanics, ASCE,

- July, pp.529.
3. Covarrubias.S.W. (1969), "Cracking of Earth and Rockfill Dams", Harvard Soil Mech. Series No. 82.
4. Hong S.W., Sohn J.I., Bae G.J., Na K.J.(1990), "A Case Study on Differential Settlement of Rockfill Dam", Proceeding, 16th ICOLD, pp.98-113.
5. Knight D.J., Worner N.M., McClung J.E.(1982), "Material and Construction Methods for a Very Wet Clay Core Rockfill Dam at Monasavu Falls, FIJI", ICOLD, Rio-de-janeiro, pp.293-303.
6. Kulhawy, F.M. and Duncan, J.M.(1970), "Nonlinear Finite Element Analysis of Stresses and Movements in Oroville Dam", Report TE-70-2 Univ. of California, Berkeley, Dept. of Civil Engineering.
7. Lawton, F.L., and Lester, M.D.(1964), "Settlement of Rockfill Dams", Proceeding, Eighth ICOLD, Vol. III, Q.32.R.2, Edinburgh, Scotland.
8. Lazanyi I. & J. Adamiak(1987), "Geotechnical Problems at the Construction of Challawa Gorge Emabankment Dam", 9th Regional Conference for Africa on Soil Mechanics Engineering, Lagos, Sep., pp321-326.
9. Sherard L. James(1973), "Embanlment Dam Cracking",pp.271-329.
10. Wong K.L, Kleiner D.E., Wood. A.M, Geary M.C., Oechsel R.G. (1992), "Design and Performance of Bath County Upper Dam and Reservoir Slopes", Stability and Performance of Slopes and Embankments-II, Vol.1, Geotechnical Special Publication No.31, pp. 371-386.