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THREE DIMENSIONAL ELASTO –PLASTIC FEM ANALYSIS OF LARGE FACING TYPE ROCK FILL DAM AND ITS SAFETY EVALUATION

Hideki Ohta
Tokyo Institute of Technology 2-12-1,O-okayama, Meguro-ku,Tokyo

Yoshihisa Uchita, Shigeru Tsuruta
Tokyo Electric Power Company,1-1-1,Uchisaiwaicho,Chiyoda-ku,Tokyo

Takeshi Ishiguro, Kazuyuki Maeda
Maeda Corporation,2-10-26,,Fujimi Chiyoda-ku,Tokyo

Introduction

A lot of facing type rock fill dams have been constructed recently. The number of large facing type rock fill dam being constructed will increase hereafter because of the reasons stated below. Unpermeable zone(core material) is not needed in this type. Recently embankments are constructed by heavy compaction rollers ,so the inclination of the slope of the embankment can be steeper. As a result, construction costs can be reduced significantly. While unpermeability of this type of rock fill dam depends on the soundness of the unpermeable wall using AFRD and CFRD constructed along the upstream slope, but the quantitative evaluation method of upstream facing wall is not suggested yet in the guidelines of the design code. The authors have suggested the evaluation method to check the soundness of center-core type rock fill dam by using the FEM analysis[1)~7)]. In this paper, a method to evaluate the soundness of large facing type rock fill dam in detail referring to our past suggested method mentioned and the case study on an actual dam after impounding in Japan are reported.

Three dimensional FEM analysis of facing type rockfill dam and comparison with measured behaviour

Facing type rockfill dam is constituted of a roller compacted coarse material and a facing wall. Compared to the center-core type dam, filling water does not get into the embankment, and the reservoiring load is added to the facing wall directly. By filling with water, upstream facing wall is deformed corresponding to the deformation of embankment. The soundness of the facing wall are judged by whether the strain of the facing wall exceeds the allowable value of facing material or not, and whether cracks of the facing wall occur or not . So it is very important to predict the deformation of embankment after filling to check the soundness of the facing wall. FEM analysis is a very useful method for this prediction. Compared to the center-core type dam, soil/water coupled analysis is not needed and total stress analysis can be adopted if construction and water filling schedules are reproduced in detail in the computer simulation, and the appropriate constitutive equation is incorporated. It is easer than effective stress analysis needed for center-core type dams. The case study to check the soundness of a facing dam by two dimensional FEM has been reported actually[8]. There are two problems even with this evaluation method. One is selection of constitutive equation of roller compacted coarse(rock) material and determination of parameters, and the other is three dimensional effect like longtitudial (light-right side) direction behavior of embankment and facing wall or the influence of foundation rock. According to this first problem, the authors have investigated the in-situ elast-plastic behaviour of roller compacted rock material under high overburden pressure, and found that the existent constitutive equation for the clay can be adopted to express this phenomenon, and also suggested that the correction of the difference of grain size between in-situ rock zone and laboratory specimen must be done. And also the authors suggested the correction method of this problem and confirmed the validity by comparing the behavior of actual dams [1)~7)]. It is possible to check the soundness of the facing wall using this investigated method and three-dimensional model.

Analytical model and conditions

YASIO Dam (AFRD: Asphalt Facing Rock fill Dam) was constructed in Japan in 1995. Its standard cross section is shown in Fig 1. In this paper, the validity of computer simulation is shown in this actual dam. Two and three dimensional models used in the simulation are shown in Fig2 with boundary conditions. Base rock foundation, dam embankment and facing wall were modeled according to Fig1. Embankment elements were added one after another according to the actual construction schedule, and

The reservoiring load was added to the upstream slope according to the actual filling schedule. For the practical use and analytical capacity, all elements were combined rigidly and the joint elements between the facing and embankment were not expressed in detail. Fig.3 shows the actual and computational step of embankment construction and water filling. As mentioned before, a constitutive equation for clay, Sekiguchi-Ohta Model[9] was used. The detail of this model and the determination procedure of input parameters are described in the reference[8]. Fig4 shows the example of determination procedure of input parameters, λ and κ considering the difference of grain size between in-situ rock zone and laboratory specimens. Parameters used in this analysis are shown in Table1 including the material constant of CFRD.

Analytical results and comparison with measured behavior

Analytical results were compared to the actual dam behavior regarding settlement and earth pressure during construction, and deformations after water impounding. The verification results are shown in Fig.5~Fig.10.In these figures, two and three dimensional analytical results and measured datum were plotted together. Fig5 shows the settlement of

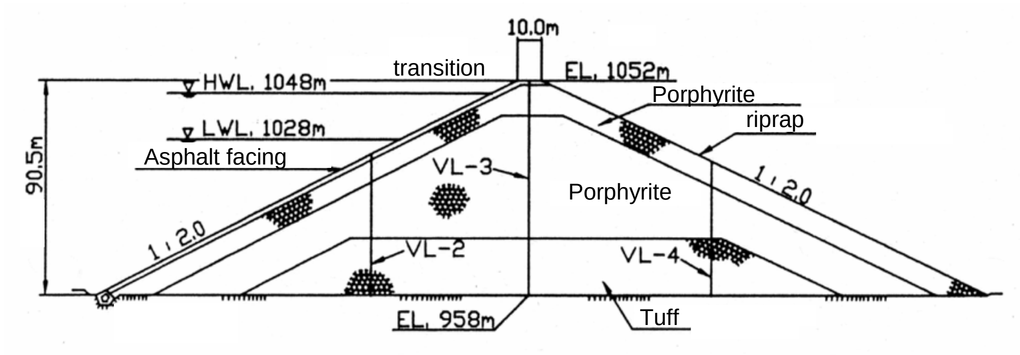

Figure 1 Cross Section of Ysio Dam

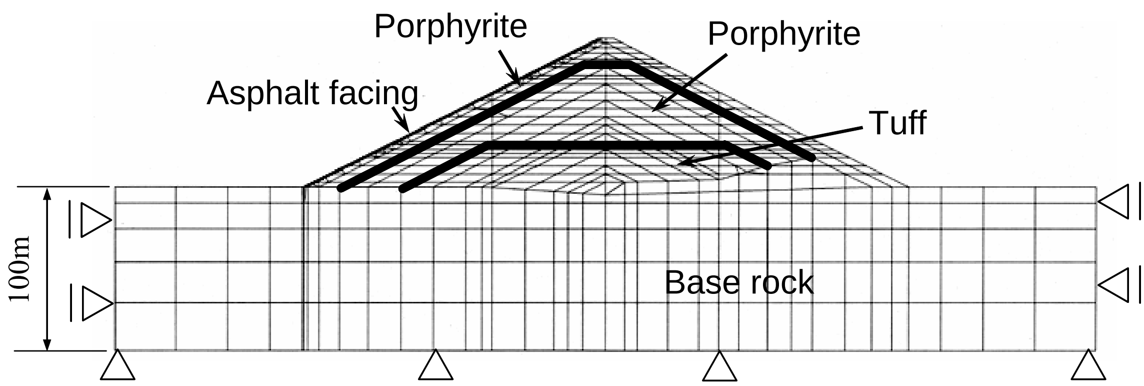

Figure 2(1) Analytical model and boundary conditions (2DFEM)

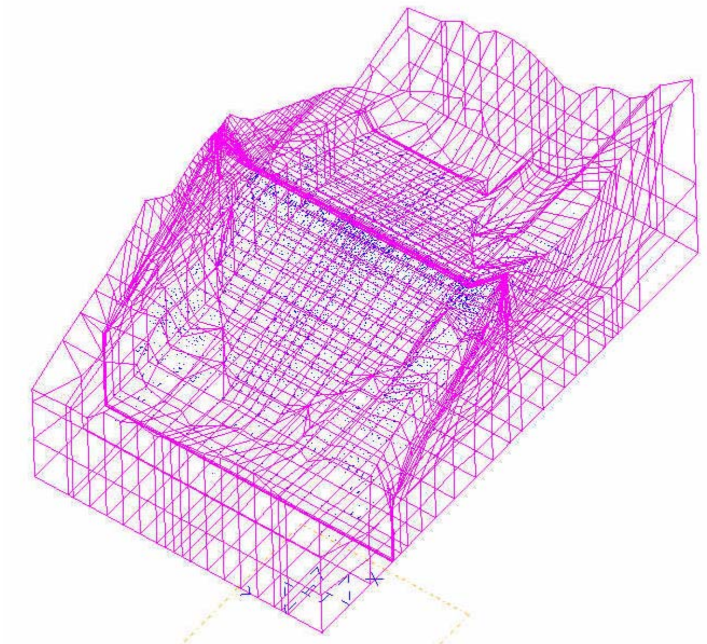

Figure.2(2) Analytical model (3DFEM)

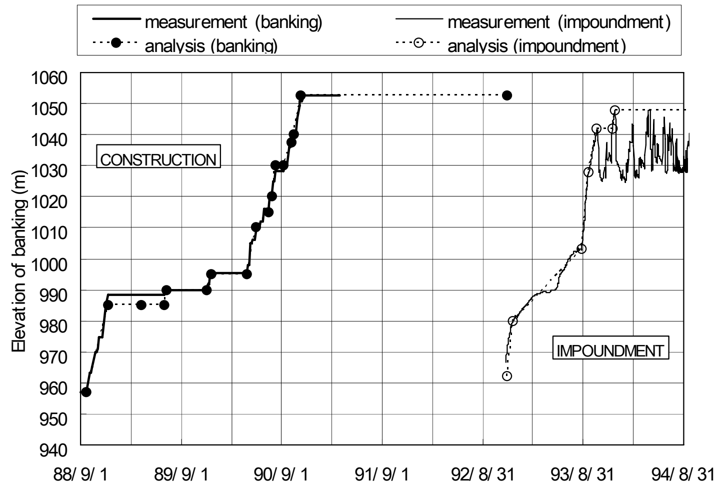

Figure.3 Schedule of construction and first impounding

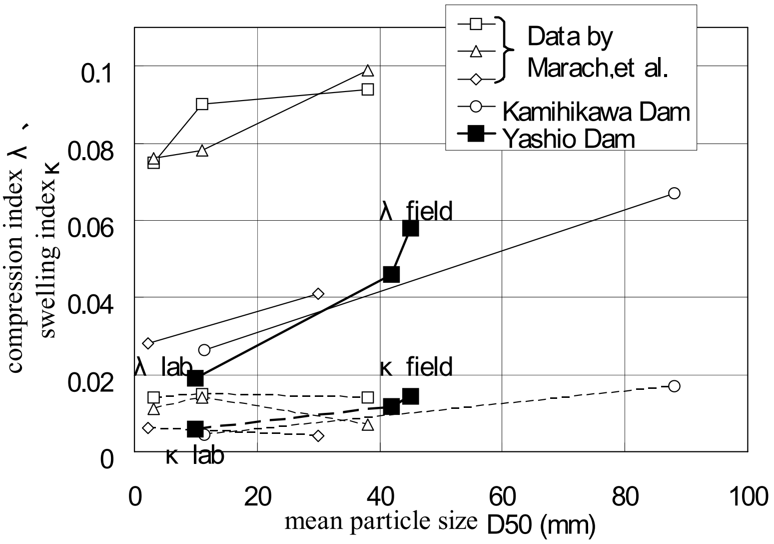

Figure.4 Method of estimating the deformation characteristics of in-situ rock zone by grain size correction

Table.1 Parameters for analysis

Elasto-viscoplastic Parameter			Transition	Porphyrite	Tuff
λ	compression index		0.046	0.043	0.076
κ	expansion index		0.012	0.010	0.017
σ_{v0}'	preconsolidation vertical pressure	kN/m^2	3000	3000	680
e_0	void ratio		0.288	0.314	0.398
σ_{vi}'	effective overburden pressure	kN/m^2	7.8	51.0	47.8
K_i	coefficient of in-situ earth pressure at rest		1.780	1.780	1.847
K_0	coefficient of earth pressure at rest		0.440	0.440	0.440
φ_f	angle of internal friction	deg.	45.9	45.9	43.6
γ_t	unit weight	kN/m^3	20.8	20.4	19.1
C_α	coefficient of secondary consolidation		0.026	0.19	3.0

Elastic Parameter			Foundation rock	Asphalt facing	Concrete facing
σ_{vi}'	effective overburden pressure	kN/m^2	352.5	351.0	352.5
K_i	coefficient of in-situ earth pressure at rest		0.5	0.5	0.5
E	modulus of elasticity	kN/m^2	1,100,000	10,000	25,000,000
ν	Poisson's ratio		0.3	0.45	0.15
γ_t	unit weight	kN/m^3	23.5	23.4	23.5

embankment after completion of construction at the three measuring apparatus (cross-arm). There is good agreement in quantity of settlement and the shape of distribution in depth. This figure shows the validity of selected constitutive equation. Fig6 shows the distribution of earth pressure in depth at the completion of construction. The results of three dimensional analysis is closer to the measured value than two dimensional analysis. This result shows the ascendancy of three dimensional analysis. Fig.7 shows the displacement after the water filling. Good agreement is also shown in this figure, a trend is shown that deformation is larger on

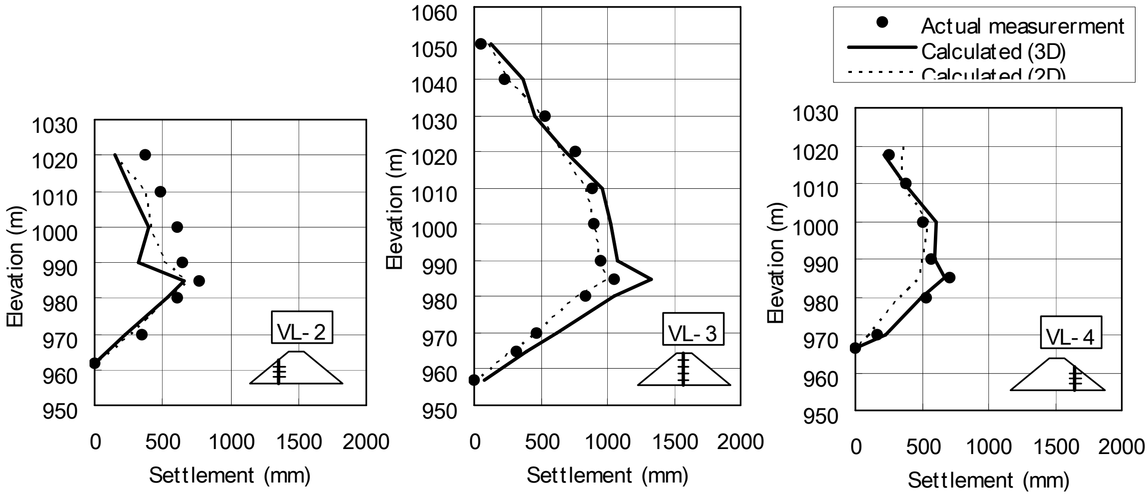

Figure.5 Comparison of measured and calculated settlements of dam embankment during construction

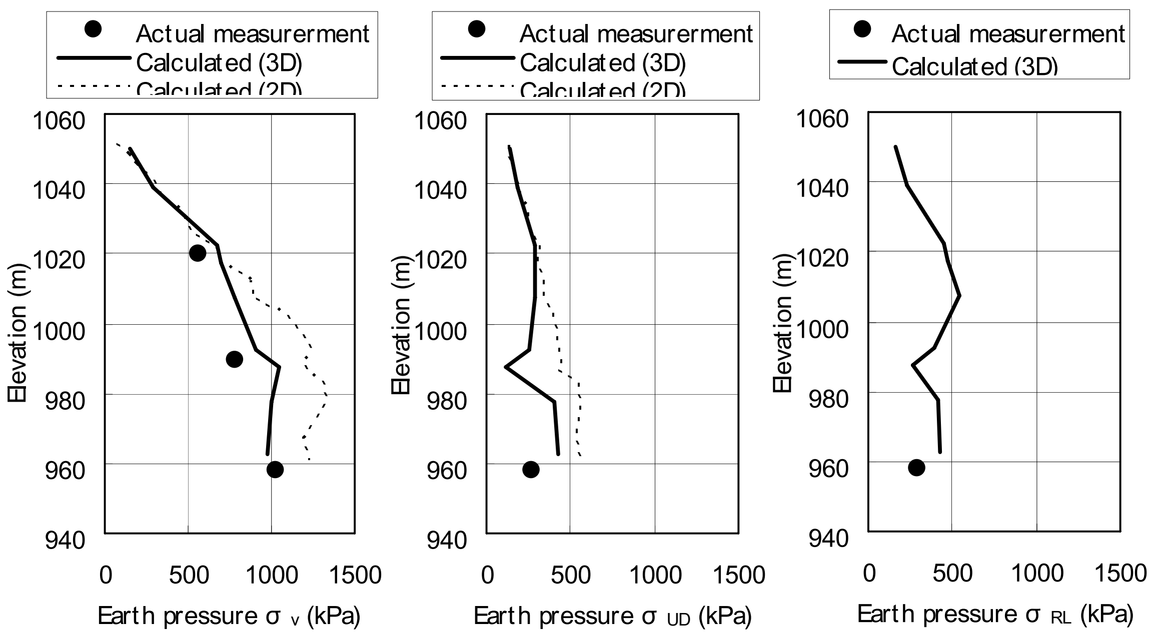

Figure.6 Comparison on the Earth Pressure Distribution

the upstream side where the water load is added. The usefulness of three dimensional analysis is clear in Fig.8, Fig.9. Fig.8 shows the distribution of the settlement along the longitudinal (left-right side) direction, and Fig9 shows the deformation of the upstream target. Fig 8 shows the distribution of settlement according to the shape of the embankment, and Fig9 shows the deformation behavior that the target deforms toward the center of the dam, and good agreements between calculated and measured deformations can also be verified. Fig 10 shows the deformation of facing wall after completion of the water filling. The measured data was calculated by special apparatus to obtain the deformation of wall perpendicular to the slope. In this figure, good agreement can be seen in the deformation shape and quantity. By these comparisons, it is clear that this computer simulation reproduces the three dimensional behavior of embankment and facing wall in practically acceptable range.

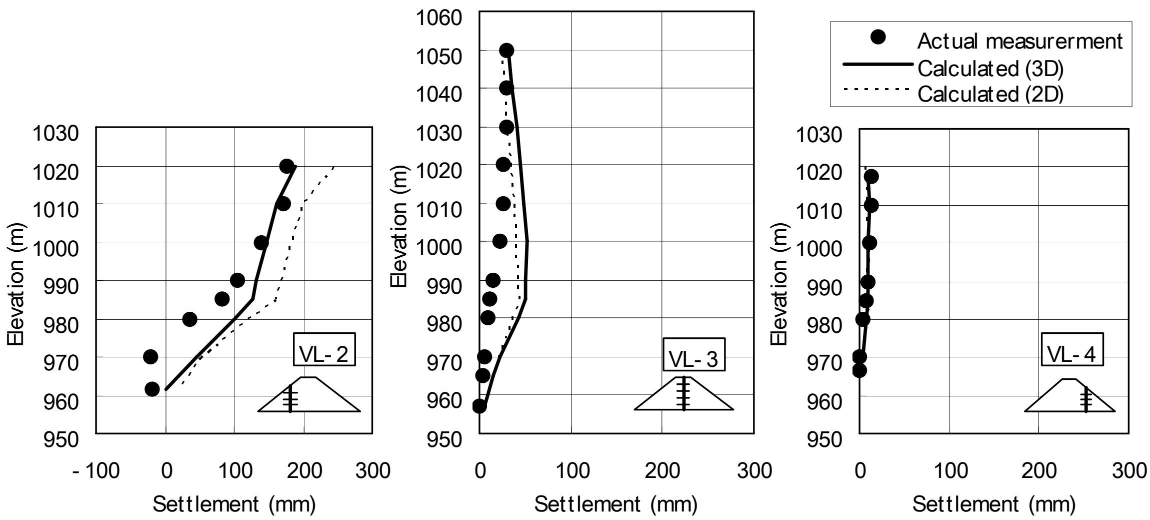

Figure.7 Comparison of measured and calculated settlements of dam embankment during first impounding

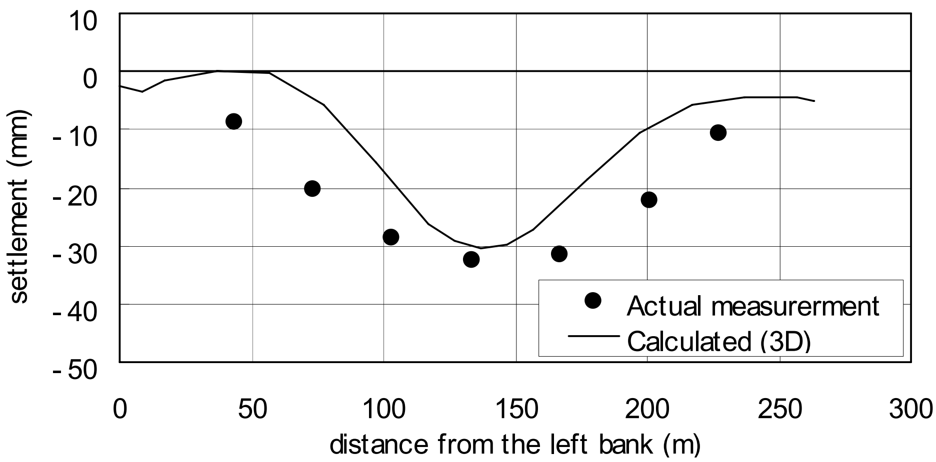

Figure.8 Settlement distribution of top of dam during first impounding

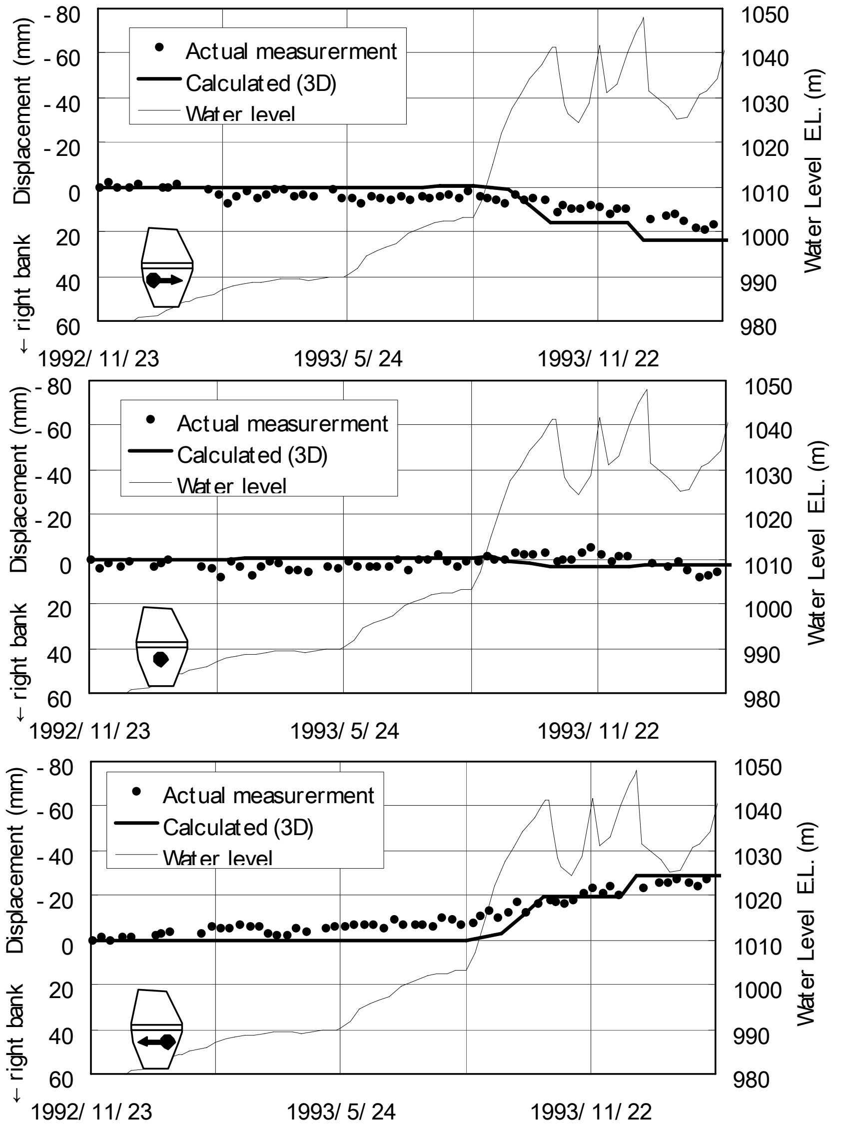

Figure.9 Displacement toward side direction during first impounding

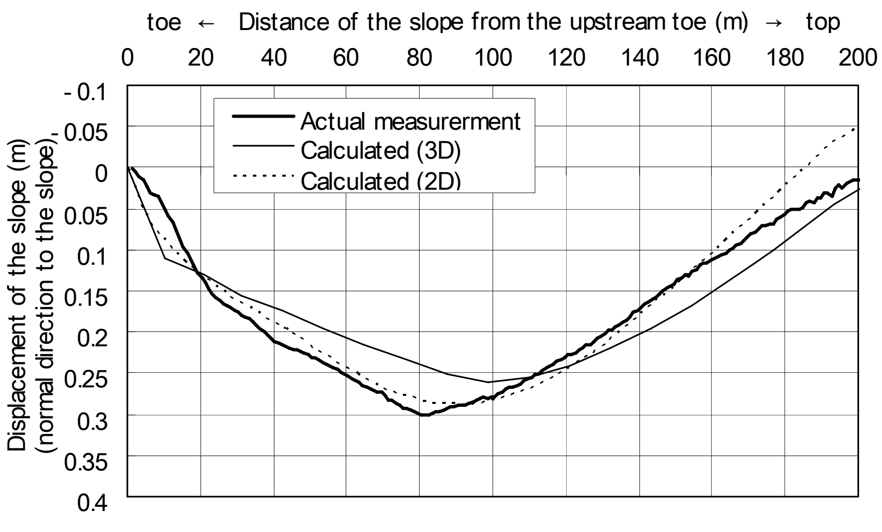

Figure.10 Displacement of the asphalt facing

Safety evaluation using the results of three dimensional analysis

Using the deformation of facing wall shown in Fig.10, its strain distribution can be calculated. The result is shown in Fig.11. Tensile strain occurs at the cut-off side and it moves to the compressional strain closer to the top of embankment. This tendency coincides to that of another dam described in past paper[7]. The allowable tensile strain of facing material(asphalt) is 0.05,and compressional 0.07. So the calculated strains are within an allowable range and the soundness of the facing wall can be verified. Fig.11 shows the distribution of strain of facing wall in an upstream-downstream direction and also restricted in the standard cross-section. This is the limit of two dimensional analysis. Again the usefulness of three dimensional analysis can be confirmed by Fig.12 and Fig.13. These figures show the complete strain-contour of upstream-downstream and longitudinal direction. X and Y axes express the coordinate (in m) of the facing . By these figures, it can be confirmed again that the strain of facing wall at any point and any direction is within the allowable range. And also it can be seen in this contour that local concentrations of strain seem to exist. Why does such a concentrations of strain occur? Fig.14 shows the three dimensional bird's-eye-view removing the embankment elements. From this figure, it is recognized that the area of strain concentration coincides to the uneven area of rock foundation. Using the three dimensional analysis, it can be tried before the start of construction to check the local concentration of facing wall, or plan the countermeasure like removal of uneven rock. Three dimensional analysis gives us a lot of useful information.

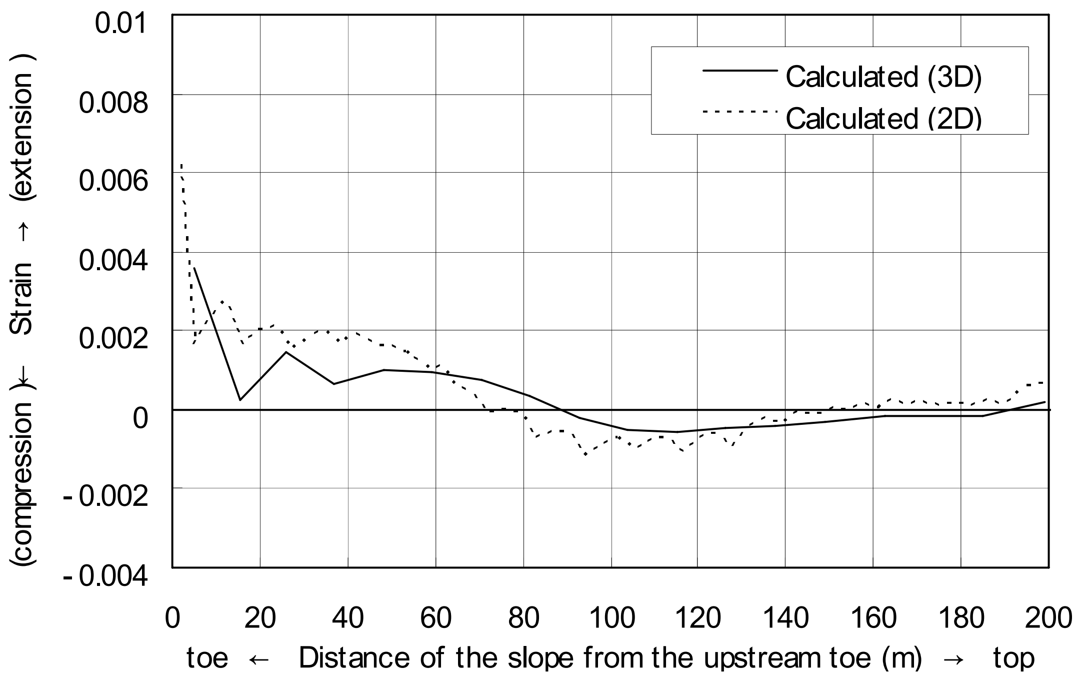

Figure.11 Strain distribution of the asphalt facing

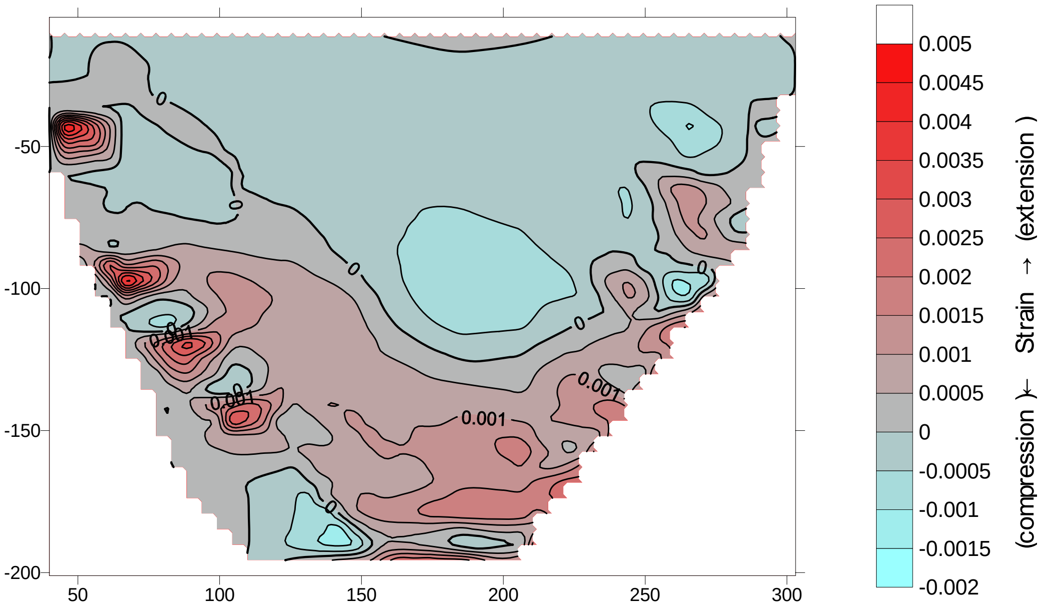

Figure.12 Strain contour of the asphalt facing (up-down stream direction)

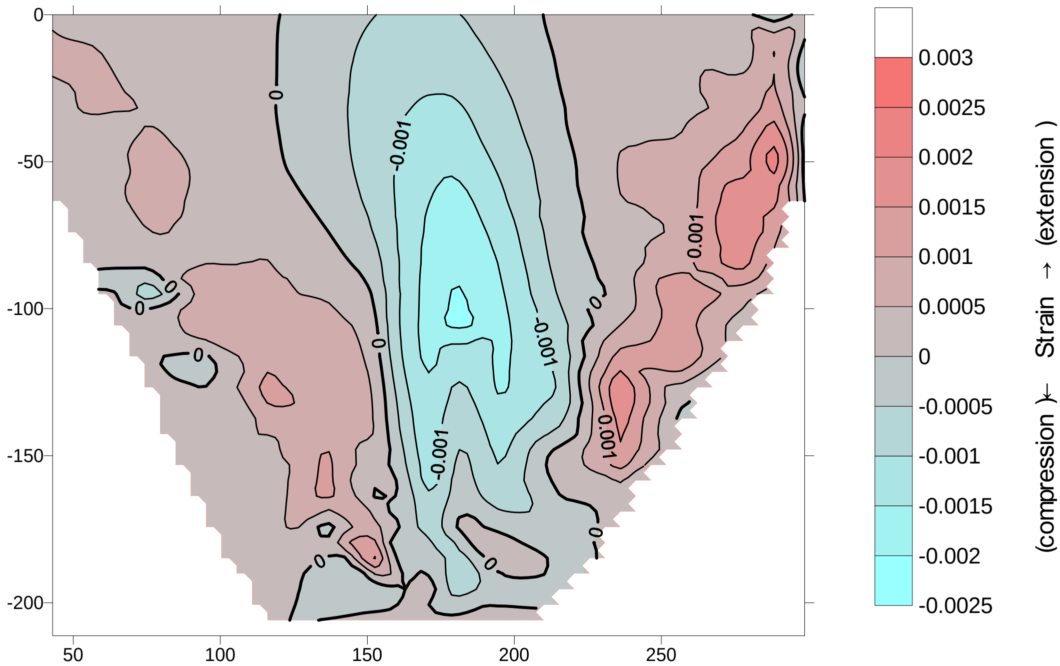

Figure.13 Strain contour of the asphalt facing (right-left bank direction)

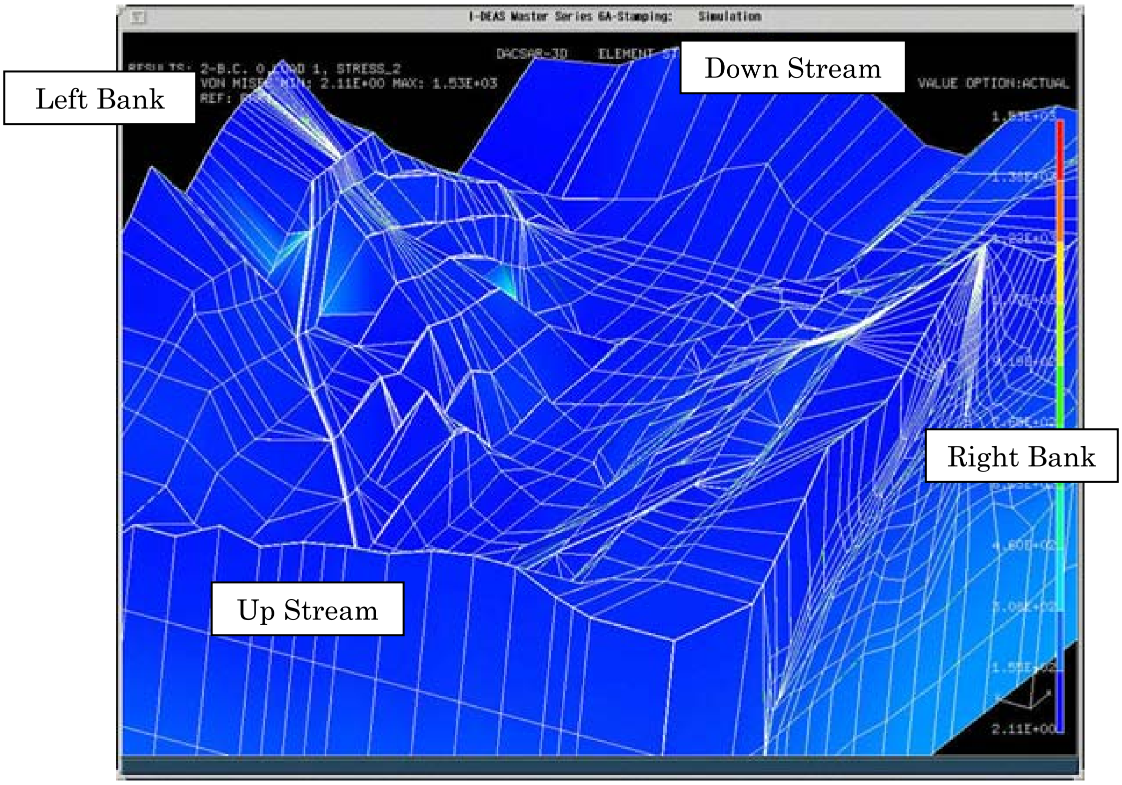

Figure.14 Foundation rock shape of Yasio Dam

Case study of CFRD

Using this analytical method, a case study (test analysis) on the CFRD was tried. Under the same conditions as the model of YASIO Dam, the properties of the facing wall were changed from asphalt to those of concrete. Fig.15 and Fig.16 shows the whole strain-contour of upstream-downstream and longitudinal direction in this case. The allowable tensile strain is 0.001 smaller than asphalt. These figures show there is a possibility that cracks of the facing wall may occur close to the cut-off and left-right side abutment at the completion of first impounding. It is impossible to compare the actual behavior, but for reference, Fig17 shows the result of observation at Cirata Dam in Indonesia. This figure shows the fact that horizontal crack occurred deeper than the depth of middle altitude of CFRD similar to the analytical result shown in Fig 15 and Fig.16 (see the areas indicated by circles). The allowable strain of CFRD is much smaller than AFRD, so three dimensional analysis using the elasto-plastic constitutive equation should be done in the design of CFRD and appropriate design of concrete execution joints or drainage layers should be planned.

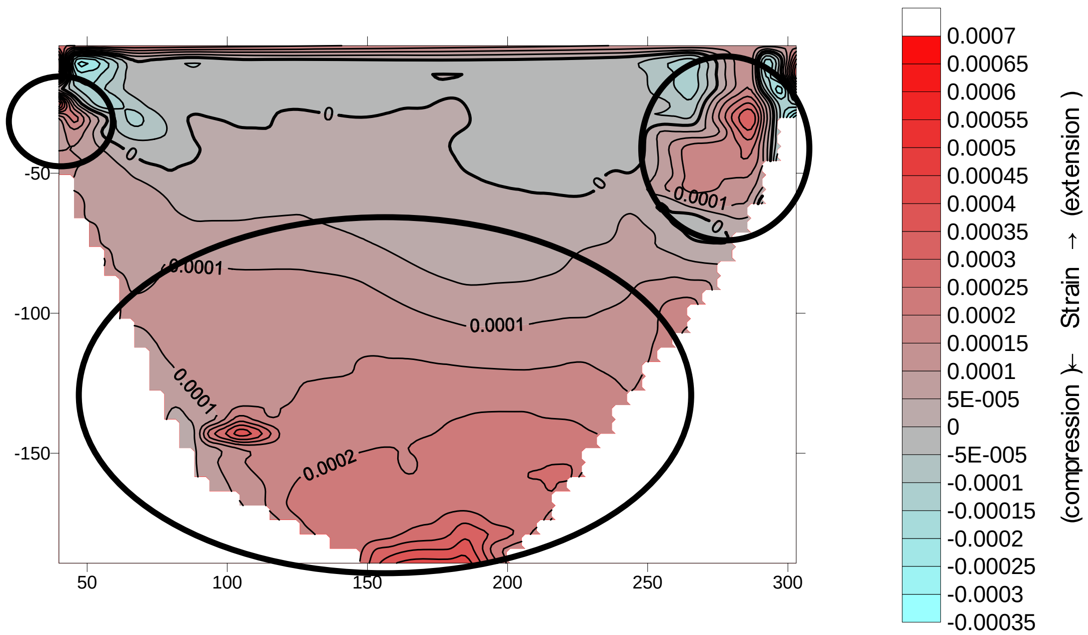

Figure 15 Strain contour of the concrete facing (up-down stream direction)

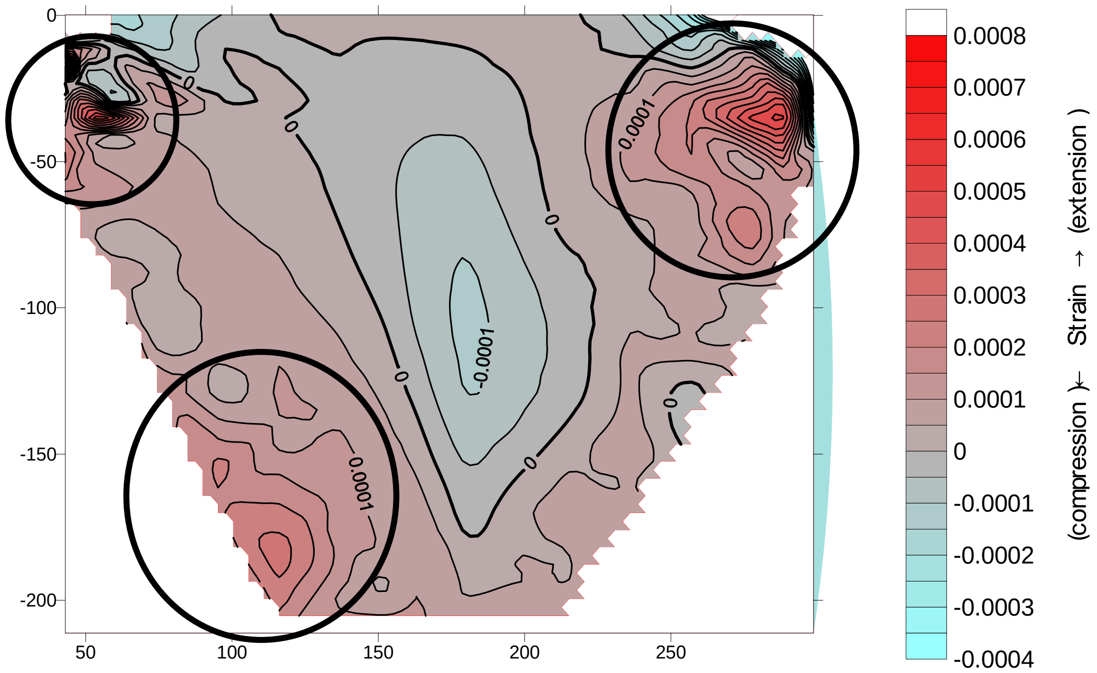

Figure 16 Strain contour of the concrete facing (right-left bank direction)

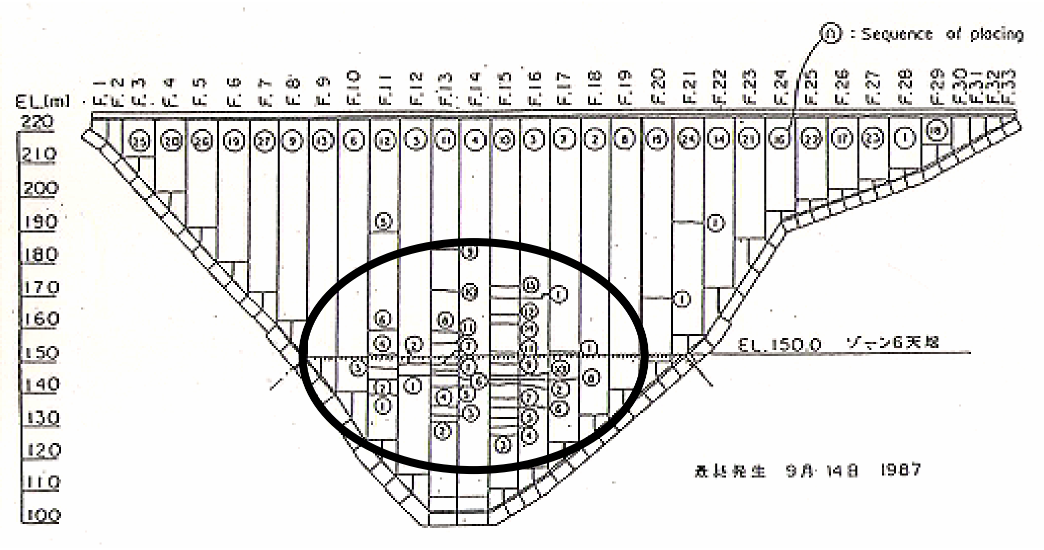

Figure.17 Observed result of cracks (Cirata Dam, Indonesia)

REFERENCE

1) Ohta,H., Yoshikoshi,H., Mori,Y., Yonetani,S., Itoh,M. and T.Ishiguro. Behaviour of a rockfill dam during construction, Proc.of 15th Inter. Conf.on SMFE, (2001)1227~1231,

2) Mori,Y., Shimokawa,Y., Uchita,Y., and Y.Nakano Analysis of a zoned rockfill dam with center core ,Q78,ICOLD,2003.

3) Hideki Ohta,Akira Takahasi,Yoshiaki Mori,Takeshi Ishiguro,Testuo Fujiyama Safety of large rockfill dams during reservoir filling, Proc. Of the 21th ICOLD,Q.86-R.,2004

4) Mori.Y,Shimokawa Y.,Uchita Y.,Tsuruta S. and Takahashi A :Uncertainty in safety evaluation of large rockfill dams during first filling, the 73th Annual Meeting of ICOLD(2005).

5) Mori,Y.,Uchita,Y.,Nakano,Y.,Yoshikoshi,H.,Ishiguro,T.and H.OhtaCompressibility and materialconstants of in-situ compacted rock material under high overburden pressure, Proceedings of the Japan Society of Civil Engineers No. 687 III-56, (2001) 233-247, (in Japanese).

6) Mori,Y., Uchita,Y., Shimokawa,Y., Yoshikoshi,H., Ishiguro,T. and H.Ohta Stress and deformation analysis of a center core-type rockfill dam during construction, Proceedings of the Japan Society of Civil Engineers No. 743/ III -64, (2003) 105-124,(in Japanese).

7) Mori,Y.,Uchita,Tsuruta,S.,Takahashi,A.Ishiguro,T.andH.Ohta:Estimation of hydraulic stability of a center core-type rockfill dam, Proceedings of the Japan Society of Civil Engineers No. 764/ III -67, (2004) 69-84(in Japanese).

8) Yasuo YANAKA Design and execution of SABIGAWA upper dam- asphalt facing type rockfill ,DAMU-HIPPONN, No.527, (1988)77-94,.(in Japanese)

9) Sekiguchi,H. and Ohta, H. Induced anisotoropy and time dependency in clay, 9th ICSMFE, Tokyo, Proc. Speciality session 9, (1977) 229~239.