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Mechanical Behavior of a Concrete Dam and In-situ Block Shear Test Analysis

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Abstract

The determination of internal stresses in a concrete gravity dam is based on the assumption of an elastic and homogeneous dam resting on rocks of similar specifications. Generally, the elasticity of rock strata may vary within very wide limits, from very rigid to very soft, and also interspersed with discontinuities. This study is to determine the variation in stress patterns of several levels in a dam and foundation, given by changes in the elasticity of the foundation. A nonlinear anisotropic model is also introduced to predict the discontinuous behavior of rock by comparing it with the in-situ test results. Design stresses in the dam and foundation of different elastic moduli have been determined by the finite element method, and from this analysis, it was found that the stress distribution in the lower 40% of a dam's height is more heavily influenced by the effects of discontinuities in the rock. The proposed model also can be used to predict the fracture behavior of rock and to observe the mechanism of rock foundation effects on the stress pattern of dams.

Key Words: stiffness, foundation rock, block shear test, crack tensor, nonlinear elastic

Introduction

The foundations on which the dams are constructed usually made of rock foundations and those are generally interspersed with joints, seams, fissures and shear zones. The stresses calculating for an arch dam shell or in a gravity dam, the assumption of an elastic and homogeneous foundation is usually made. But studies by many investigators show that the anisotropic and inhomogeneous discontinuities of rock affect not only the strength and deformation behavior of the rock mass but also the stress distribution in the lower portion of the dam. The change in the elastic property of the foundations, also, gives rise to variation in stress pattern in the dam.

The strength of rock depends in a complicated way on its structure, size, and the stress conditions. In order to evaluate the deformability of discontinuous rock mass under changing stress condition, the block shear test is performed in many cases. In this study, the nonlinear elastic model and crack tensor theory are employed in the continuous model to represent such a complicated behavior of rocks.

Many studies of the stress distribution, using elastic theories, have been presented for gravity dams or similar massive structures resting on rock foundations. Zienkiwicz and Gerstner, 1961[5], Zienkiwicz and Cheung, 1965[10] and Alessandro, 1974[11] discussed that the complete avoidance of tensile stresses in each

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structures is impossible; and to determine whether the dam's safety is possible or not, it is necessary to depart from the conventional criterion of no tensile stress. Moreover, Varshney, 1974[4] also illustrated the elastic stress distribution in a layered half plane stress and plane strain conditions. Most of these studies made the assumption within a layer and the rock mass is regarded as homogenous transversely isotropic elastic medium. The plain of isotropy is parallel to the foundation boundary and the layers are firmly connected together along the boundaries. Nevertheless, Maury, 1970[12] founded that the stress distribution is greatly influenced by the layers in between and differences in the nature and intensity of stress between discontinuous media and the continuous media used in the rock foundation analysis.

The behavior of jointed rock masses is dominated by the behavior of rock joints. A number of experimental studies have been conducted to understand the behavior of natural as well as artificial joints. Joint in rock masses can vary widely in the physical state and mechanical behavior. They can be fresh or weathered, asperities matching or mismatching, filled or unfilled with gouge material. This is so because the failure takes place through the infilling material and the characteristics of the joint walls play an insignificant role.

In the past, a number of empirical relationships have been proposed relating the strength and dilation of rock joints. Notable amongst these studies are due to Goodman, 1974[14], Barton and Choubey, 1977[13], Bandis et al., 1981[8]. More studies have been concerned with strength than with deformability.

In this study of foundations for concrete dams, two problems arise; firstly to investigate the effect of the stiffness of the rock mass on the behavior of the dam and the contact area of foundation, and secondly, to introduce the nonlinear anisotropic model for fractured rocks by comparing with the in-situ test results.

Methodology

The gravity dam design is based on the "gravity analysis" under the two main assumptions, i. e., the vertical stress is linearly distributed on foundation rock and elastic properties of the foundation and the concrete of dam are the same[2]. This study is to determine the influence of rock properties on stress behavior of a concrete gravity dam.

In the first part of study, stress distribution is determined due to the different Young's modulus of foundation rock based on the linear elastic behavior of stress analysis. The finite element method analysis is used for this analysis. The horizontal and vertical stress distributions are to be calculated in rigid foundation case $E_c/E_r < 1.0$, in common assumption case $E_c/E_r = 1.0$ and in soft foundation case $E_c/E_r > 1.0$, where E_c and E_r are the Young's modulus of the concrete and foundation rock, respectively. The stresses distributions are determined by using LUSAS program, which is commercial software.

Secondly, the crack tensor theory originally proposed by Oda, 1986[7] and the nonlinear elastic model of single fracture proposed by Baton and Bandis, 1981[8] are used together in the mechanical model. This model is applied to the in-situ tests that investigate the stiffness of the foundation properly and compared them with measured values of real data of experimental site.

Through the comparison, the behavior of the rock mass can be examined. The following cases are carried out;

Case1 Elastic condition

Case2 Elasto-plastic condition (Von Mises)

Case3 Nonlinear elastic with anisotropy

Theory[7)]

The Barton-Bandis (*BB*)[8)] model represents the change in the normal and shear stiffness due to the stress change by using *JRC* (Joint Roughness Coefficient) and *JCS* (Joint Compression Strength). By using the JRC^I_n and JCS^I_n (MPa) which is the mean values for each set *I*, the initial normal stiffness of joint, K_{ni} (MPa/mm), is calculated for each set I,

$$K^I_{ni} = -7.15 + 1.75 JRC^I_n + 0.02\ (JCS^I_n / a^I_j) \tag{1}$$

Where a_j (mm) is the initial joint aperture given by

$$a^I_j = \frac{JRC^I_n}{5}\left[0.2\frac{UCS}{JCS^I_n} - 0.1\right] \tag{2}$$

where *UCS* is the uniaxial compression strength of rock. The normal stress, σ^I_n to each set is calculated by $\sigma_{ij}\, n^I_i n^I_j$ in which σ_{ij} is the calculated stress of each element, and n^I_i is the mean unit normal vector of each set. The normal stiffness of the joint is revised by using the normal stress at each time step.

$$K^I_n = K^I_{ni}\left[1 - \frac{\sigma^I_n}{V_m K^I_{ni} + \sigma^I_n}\right]^{-2} \tag{3}$$

and the shear stiffness is obtained from

$$K^I_s = \frac{100}{L^I}\sigma^I_n \tan\left[JRC^I_n \log_{10}\left[\frac{JCS^I_n}{\sigma^I_n}\right] + \phi^I_r\right] \tag{4}$$

In equation (3), V_m is assumed to be the same as a^I_j of equation (2). L^I is the mean fracture length of the set (mm) and ϕ_r is the residual friction angle.

By using $h^I = K^I_n L^I$, $g^I = K^I_s L^I$ the elastic compliance C_{ijkl} and the hydraulic conductivity tensor k_{ij} are calculated by

$$C_{ijkl} = \Sigma_i \left[\frac{1}{h^I} - \frac{1}{g^I}\right] F^I_{ijkl} + \frac{1}{4g^I}\left(\delta_{ik}F^I_{jl} + \delta_{jk}F^I_{il} + \delta_{il}F^I_{jk} + \delta_{jl}F^I_{ik}\right) \tag{5}$$

in which the summation is carried out for sets. The δ_{ij} is the Kronecker delta. F_{ijkl} and F_{ij} are obtained from

$$F^I_{ijkl} = \frac{\pi}{4} L^3 n^I_i n^I_j n^I_k n^I_l \rho^I \tag{6}$$

$$F^I_{ij} = \frac{\pi}{4} L^3 n^I_i n^I_j \rho^I \tag{7}$$

in which the summation is carried out for the fractures included in each set. By using the fracture geometry information, the fractures are realized with the random number. The value of F_{ijkl} and F_{ij} are determined and not changed through the analytical process.

$$\left[\frac{1}{2}T^{-1}_{ijkl}\left(u_{k,l} + u_{l,k}\right) + \chi\, T^{-1}_{ijkl} C_{kl} p_{,l} h\right] + pb_i = o \tag{8}$$

where $T_{ijkl} = (M_{ijkl} + C_{ijkl})$, in which M_{ijkl} is the elastic compliance of the rock matrix.

$C_{ij} = C_{ijkl}\,\delta_{kl}$.

In the case where a failure criterion is considered, *JRC* becomes 1 and *JCS* is reduced to one tenth of the original value. The failure criteria are assumed to be Mohr-Coulomb criterion.

The body is divided into smaller three-dimensional regions. The equivalent of a triangular element is a tetrahedron in 3D and that of a four noded rectangle is the eight-noded brick element.

Dam section and load condition

Dam section

For the purpose of analysis a gravity dam section of 75 m height and downstream slope of 0.79:1 is adopted. No fillets are provided at the heel and toe of the dam, because of any particular shape of fillets could have affected the stresses in a particular manner and has local effect only. The FE mesh used is shown in Fig. 1.

Parameter description

The elasticity of dam material is taken as $2 \times 10^7 \mathrm{kN/m^2}$, which is a common average figure. The Poisson's ratio is adopted as 0.20. The unit weight of concrete is 24 $\mathrm{kN/m^3}$.

Loading condition

In this study, four types of loading condition are applied, these are water pressure, self-weight, uplift pressure and sedimentation loads. Other loads as initial stress in the rock mass and thermal stresses are not considered. These stresses differ from one rock to another and one site to another.

The dead weight of the foundations has been excluded and the gravity load is distributed equally to the nodes of the element and that is defined as (*CBF*). The water pressure and sedimentation load acting on the free surface is replaced by statically equivalent concentrated loads acting at the nodal points and which are defined as Face Loads (*FLD*) in program. No tail water loading is considered in all the cases. It is reasonable to assume that initially a hydrostatic pressure distribution exits within the original riverbed. The full intensity is taken at heel reducing to zero at the toe. This condition is referred to as full uplift condition and that is an extreme condition. Earthquake loads are not considered because of the dynamic analysis is beyond the scope of this work.

The finite element mesh diagram and load distribution diagram are shown in Fig. 1. The grid consists of 326 elements and 1059 nodes. The rock area covered by the grid extended about 3.5 times of the dam height on either side.

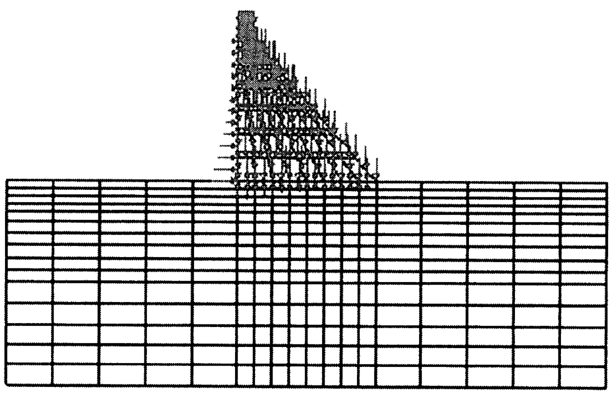

Fig. 1 Element meshes and load distribution diagram

Case study

The following cases are analyzed for a wider range of foundation i. e. elasticity ratio, E_c/E_r from 0.1(E_r= 2×10^8kN/m^2) through 0.9 (E_r=0.222×10^8kN/m^2), 1.0 (E_r=2×10^7kN/m^2) and 2.5 (E_r=0.8×10^7kN/m^2), 4.0 (E_r=0.5×10^7kN/m^2), and 10.0 (E_r=2×10^6kN/m^2). This covered from very rigid to very soft foundations.

Test-site and testing procedure

In this study, the mechanical properties of rock data are based on the geological investigation of experiment site of Ohshida dam, on Mabuchi River in Iwate prefecture. To understand the deformability and strength of the dam foundation, the block shear tests is carried out. The tests are conducted by digging the different place of trench in every year (1996~1998).

Present analysis, the 1998 trench data is used. This trench is located at the elevation of 392.5 m on the left side of the dam and the size of the trench is 15m length, 2m width and 2.5m depth. Geology is a bedded chart of Hiranuka stratum, which has an orientation of N 20° ~30° , W75° ~80° . The fractures are mostly parallel to the dam axis and steep. The rock classifications present following general tendencies; the rock mass at the trench is mainly composed of classes named as CH and CM. In 5m from the mountain side, the rock was classified as CH class and in the other part, it was CM class.

Testing procedure

Firstly, the concrete blocks of approximately 0.6 m × 0.6 m × 0.3 m in sizes are constructed at the vertical stresses of 0.6, 0.8, 1.0 and 1.2 MPa. Then, 15° inclined stresses is loaded in step by step . The block shear test is examined at the CM class.

Data preparation for analysis

The block shear test is applied at four locations, which are S1, S2, S3 and S4. The area of shearing for each test is 0.6m×0.6m. The geometric information of the discontinuities such as fracture length and orientation is inferred from the geological investigation results. Table 1 indicates the geometric information used in the analysis. The analysis for crack tensor is based on crack geometry in normal direction of dip angle, length of fracture and fracture intensity due to their orientation. The other data, *JCS*, *JRC*, ϕ_r and *UCS* are assumed as shown in Table 1. According to the orientation, strike angle is grouped and there are

Table 1 Fracture geometry information

No. set I	1	2	3	4	5	6	7
Dip direction	18	180	153	117	245	350	315
Dip angle (degree)	63	63	22	45	80	55	70
Length (cm)	200	200	200	200	200	200	200
Residual angle (degree)	33.3	33.3	33.3	33.3	33.3	33.3	33.3
JRC	20	20	20	20	20	20	20
JCS (MPa)	100	100	100	100	100	100	100
UCS (Mpa)	100	100	100	100	100	100	100

7 groups in this analysis and fracture intensities are determined by strike angle frequency in each group as shown in Table 2.

Table 3 shows the mechanical properties of rocks and concrete. Those of rocks are estimated from the in-situ shearing test, while those of concrete are obtained from literature.

Result and Discussion

In general, the elasticity of rock strata may vary within wide range, from very rigid to very soft. More detailed study is carried out for the variation of vertical stress pattern on horizontal plane concerning with the change in the elasticity of foundation. The result obtained by this finite element method is evaluated with the results from "gravity analysis" method and other numerical studies also.

As shown in Fig. 2, the vertical stress is not linearly distributed at the part of less than 33 m of height of dam section. On the other hand, the part higher than 45 m of the dam has the linear stress distribution along horizontal direction, this tendency coincides with classical theory. If the concrete-dam body and foundation rock are considered as the same Young's modulus of elasticity, no tension stress is occurred in the dam. In calculating stress in gravity dam, the foundation rock is assumed to be comparable in elastic properties with the concrete of the dam body, however, in reality, the rock behavior effects on the variation of stress and it should be recognized.

The results in terms of vertical stress history with respect to horizontal surface of contact plane for variation of foundation's elasticity are shown in Fig. 3. Pronounced changes occur in the contact stress distribution, as the foundation varies from very hard to very soft. It may be seen that a harder foundation reduces the compressive stresses in downstream (toe portion) and increases the tensile area in upstream (heel portion). The reverse happens when the foundation is relatively soft. Similar problem of dam design together with its foundation was studied by the application of the probability method of least squares on digital computer by Kharkov,1964[6]. With this finding, it can be inferred that the stresses in the lower 30% of a dam's height agreed with the classical theory. For comparison of different gravity dams, actually

Table 2 Fracture density (Num/volume (cm^2)) for each set ($x10^{-3}$)

No. set	1	2	3	4	5	6	7
S1	5.3	1.1	0.6	0.3	0.3	0.3	0.3
S2	6.1	0	0	0.3	0.3	0.2	0
S3	5.0	0.8	0.8	0	0	0.9	0
S4	5.0	1.1	0.6	0.3	0.3	0.3	0

Table 3 Material properties of rock and concrete

Material properties	Rock	Concrete
Young's modulus (MPa)	6,000	25,000
Poisson's ratio	0.3	0.167
Density (g/cm^3)	2.2	2.3

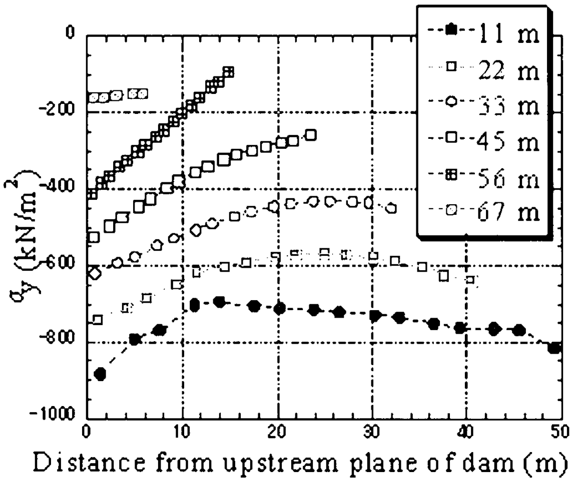

Fig. 2 Vertical stress distribution in different height of the dam ($E_c/E_r = 1$, positive; tension) (1 Mpa = 1000kN/m^2)

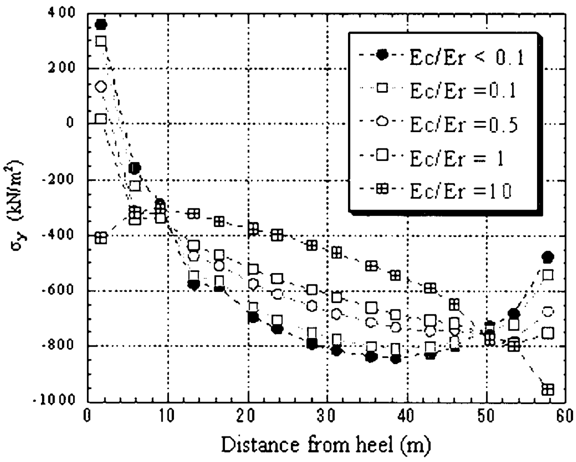

Fig. 3 σ_y -distribution of the first layer of foundation (positive; tension) (1 MPa = 1000kN/m^2)

measured data was collected for the analysis, and plotted by Varshney, 1973[3]. According to his plotting, it is obvious that there is no regular pattern of stress distribution and the variation of vertical stress is non-linear.

From the results of analysis, it can be inferred that the maximum vertical stress in the contact rock surface is occurred at the two third of dam base in case of very rigid foundation, i. e. $E_c/E_r < 1.0$. When the foundation rock becomes softer than the dam, i.e. $E_c/E_r = 10$, the maximum vertical stress arise at the downstream as shown in Fig. 3. The maximum stress in concrete is not at downstream in harder foundations. The maximum stress is higher than downstream stress by about 11 percent in the case of very rigid foundation to 60 percent in the case of very soft foundation. The maximum stress in the rock is shown as the E_c/E_r ratio increase i. e., the rock is implemented as softer, the maximum value and downstream stress approach more nearly.

The geometry and finite element mesh are shown in Fig. 4. The mesh diagram is composed of two materials such as upper portion represents for concrete block and another is for rock foundation. In rock block, bottom boundary and lateral boundary are restricted for displacements. At the top boundary, vertical loads are imposed. For the displacement variable, lateral boundary are fixed in the horizontal direction and at the bottom boundary, displacement is restricted for both directions. The concrete block of approximately 0.6m×0.6m×0.3m in size are constructed. Here, top boundary and horizontal boundary are restrained for displacement in both directions and at bottom boundary, vertical displacement is free in concrete block. The rock block is assumed as CM class and block shear test is implemented.

Fig. 5 shows for the relation of shear stress with respect to displacement in shear direction. Although this study is to predict the complicated behavior of rock in nonlinear elastic condition, however, Case 1 is implemented in the elastic condition for making the comparison among these conditions and therefore, the characteristic of this variation for nonlinear failure criteria can understand easily. Under elastic condition, shear stress and shear displacement varies linearly. For the case of elasto-plastic condition, Case 2, the yield stress is determined in Von Mises criteria. Here, shear stress and displacement varies as a curvature. The

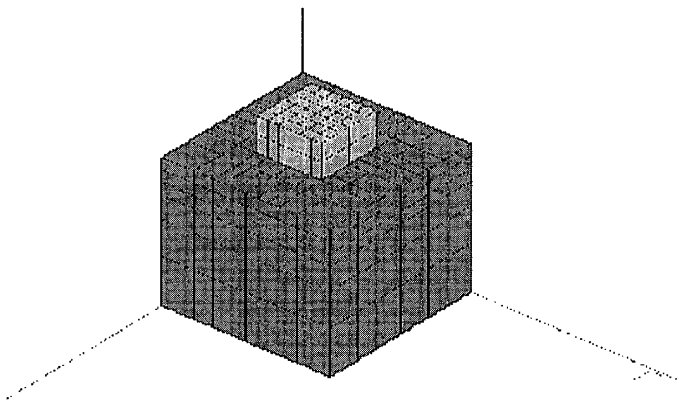

Fig. 4 Mesh diagram

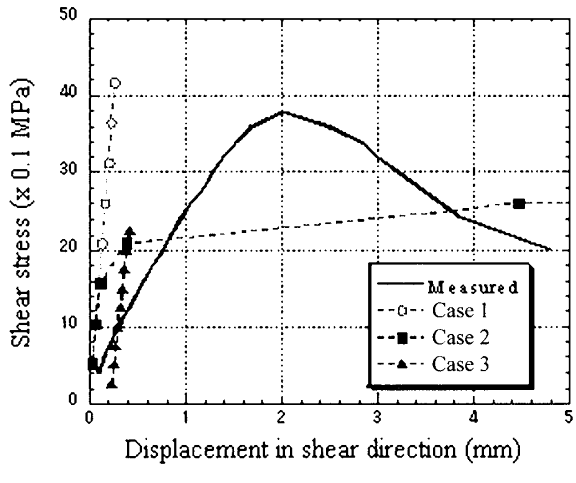

Fig. 5 σ - displacement in shear direction

result for the simulation with Case 3 is nearly the same as elastic behavior because of failure is not occurred in this case. It is obvious that the first jump is occurred in all cases. This means all displacements not start from zero displacement probably because of drastic change of stiffness due to first loading. If more fine loading condition is set, probably the displacement may be started from zero displacement.

From this graph, recognizing that the starting point of curvature for simulation of Case 2, i. e, elasto-plastic condition, the detailed study for that point is demonstrated in Fig. 6. For the case of elasto-plastic condition, Case 2, the yield stress for Von Mises criteria is set at the c value of Mohr-Coulomb criteria of CM class rock. This figure is plotted the relation between shear stresses and normal stresses for starting point of curvature of Case 2 and Case 3. From this graph, it can be noted that the starting point of curvature of shear stress and displacement by Case 3 using 1.2 MPa loading coincides with the failure criteria measured by in-situ test. On the other hand, the elasto-plastic model (Case 2) gives more weak yield point. Although, it is difficult to realize the real phenomena of failure in contact surface, this analysis can be understand that anisotropic condition of rock influences to stress-strain behavior of concrete.

The results of the simulation with Case 3 condition are depicted in Fig. 7 for the relation of vertical displacement and shear displacement on contact surface of concrete block and rock foundation. This problem is analyzed for loading condition of 0.6 MPa and 1.2 MPa. In all cases, vertical displacement illustrates negative value which means that contact surface forms settlement in concrete block. From the graph, it can be seen that the result of simulation is bigger than the result of in-situ (experiment) although the tendency is the same.

From this analysis, tension stress is not occurred in the concrete of the dam for all cases. On the other hand, the tension stress at just under the heel increases in hard foundation that is similar to the results found by Varshney, 1974[4] and Zienkiewicz, 1961[5]. Thus, it is found that the estimation of the stiffness of foundation is very important for the design of the concrete dam.

As discussed above, the nonlinear anisotropic behavior may have a great influence on the rock property

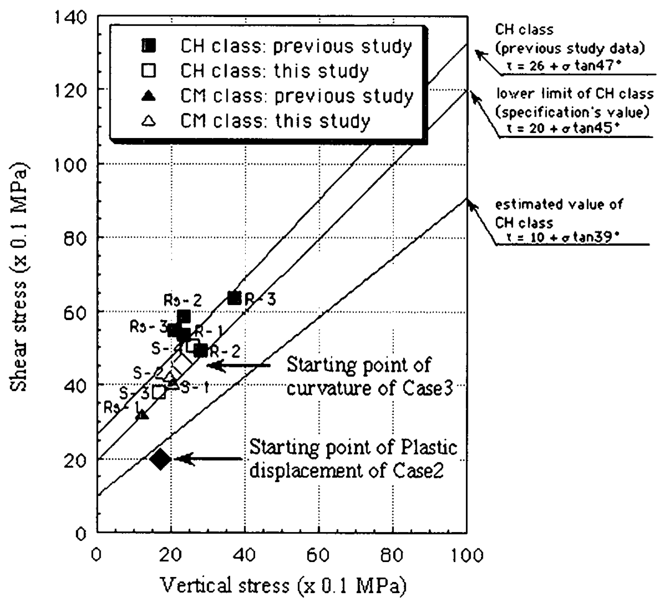

Fig. 6 Shear and vertical stress relationship

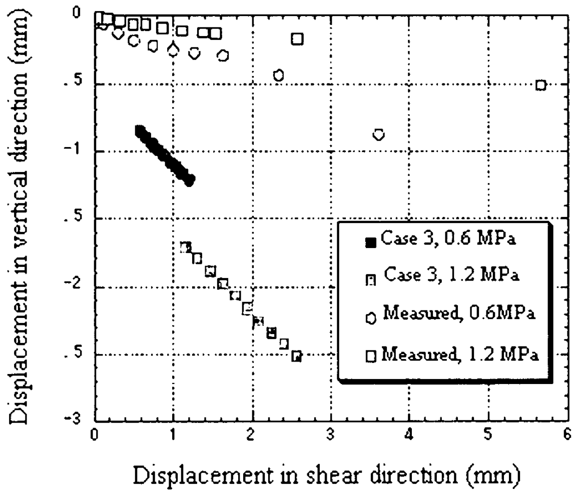

Fig. 7 Vertical vs. shear displacement of concrete block

and the proposed model can be used to predict this behavior of rock.

Conclusions

(1) The stress distribution in the lower 40% of a dam's height differs considerably from that given by the classical gravity analysis and is very much influenced by the nature of rock fabric.

(2) The stress distribution in the upper part greater than 60% of a dam's height is nearly the same as given by gravity analysis. The variation in rock properties has little effect on the dam stresses in the upper region.

(3) The tension stress region has been found near the heel of dam in all cases of rigid foundations ($E_c/E_r < 1.0$). This tension stress is not indicated by the gravity analysis. But tension stress is not occurred with the soft foundation in the upstream zone. To improve the stress pattern and reduce high tension, foundation treatment in the upstream region is desirable and the tensile stresses can be controlled to some extent by taking suitable measures in foundation treatment.

(4) The vertical compression stress at the downstream is smaller than that obtained by classical gravity analysis, i. e., compression stress of 988.17 kN/m^2, while that at the upstream is larger than that of gravity analysis; 133.452 kN/m^2.

(5) In this paper, the newly developed nonlinear anisotropic with failure criteria model for fractured rock is introduced and this applicability is examined by comparing with the in-situ test results. From this comparison, it can be concluded that the proposed method can predict the nonlinear anisotropy behavior of rock even it is difficult to realize the real phenomena. This is probably because the actual foundation has smaller stiffness than numerical model.

(6) Failure is not occurred at any set, while the resultant relation between shear stress and displacement of Case 3 shows the curvature at the last loading step for the case of vertical load of 1.2 MPa.

(7) The starting point of curvature of shear stress and displacement by Case 3 for 1.2 MPa loading coincides with the failure criteria measured at in-situ test. On the other hand, the elasto-plastic model (Case 2) gives weaker yield point.

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コンクリートダムの力学性状と現位置ブロックせん断試験解析

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コンクリート重力ダムの力学性状と現位置せん断強度パラメータ及びロックマスの変形特性との相関性を明らかにすることは堤体の設計と安定性の検討に際して欠かすことのできない検討事項である。

本論文では堤体と基礎の弾性係数を変化させることによって，堤体内の各レベルにおける応力パターンの変動を求め方法を示している。不連続性岩盤の挙動を予測するために非線形異方性モデルを採用した。異なる弾性係数を有する堤体と基礎に対する設計応力は有限要素法を用いて計算し，それらの数値を現位置試験の測定値と比較した。その結果，ダム高の下からほぼ 40%の位置における応力分布は岩盤の不連続性の影響を大きく受けることが明らかとなった。

提案したモデルは岩盤の破壊性状を予測し，また，岩盤性状が堤体に与える影響を調べる上で有効であると考えられる。