Accurate Survay Effect of Jointing On Bearing Capacity for Rock Foundation (A Case Study of Safa Dam in Kerman Province, Iran)

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Paper Reference Number: 07-08-5252
Name of the Presenter:

Abstract

The effect of discontinuities in bearing capacity of the rockmass depends on its orientation, surface conditions, and infillings. This study was conducted to calculate bearing capacity of the safa dam's rockmass using analytical and numerical techniques. In analytical technique using Serrano & Olalla method and in numerical technique using ABAQUS software. In per second methods, stress analysis was performed in two states. In one state, assuming a continuous environment for the bedrocks, the analysis was performed for the bedrock considering rock mass geotechnical parameters. In the second state, the analysis was performed considering two joint set in the bedrock as weak close planes with respect to the geotechnical of the intact rock. For these two states, the stress and safety factor, compressional and tensional, in turn, ultimate and allowed bearing capacities were estimated. Finally, the ultimate and allowed bearing capacities obtained through the different approaches were compared. The difference between the obtained allowed bearing capacities in two mentioned states implies the considerable effect of joints and cracks conditions on bearing capacity of the foundation in this project.

1. Introduction

Rock mass properties depend on discontinuities’ conditions as well as nature of the rock material. However, various rocks are suitable for majority of foundation types and rock foundations provide a safe ground to construct structures on them; providing degradation, weathering, joining, and faulting in the rock mass is in an acceptable level. The major difference between intact rock and rock mass units completely differentiates their behaviors. Due to such differences rock masses, unlike intact rocks, experience considerable deformations and settlements even when subjecting to small loads. The loads induced by the ordinary and light structures are as much as large that even relatively weak rocks can tolerate them. On contrary, once rather sound rocks are subjected to the stresses induced by huge structures such as dams, skyscrapers, and bridges abutment, they may reach to their ultimate bearing capacity. So, this highlights the necessity of performing accurate studies in bedrocks of such structures and calculation of bearing capacity in these foundations.

2. Introduction of rock mass in Safa Dam site

Safa reservoir dam is located in in Kerman Province, in 30 km NE of Baft town, Iran. The dam is constructed in intersection of Rabar and Roudbar rivers. This is an embankment dam with a vertical clay core and reservoir with capacity of 126×10^6 m^3. The height of dam crest from river bed is 86 m while the crest length is 1711 m. The rock units in the site mainly consist of sequences of marl and mudstone and fine sands with silty-marl and silt formation. Based on field observations of the bores and exploration galleries in both banks of the dam, it was found that rock mass of the Safa Dam site has abundant of joints and discontinuity surfaces, where are mainly made of soft clay or gypsum infillings which decrease strength parameters of the rock mass. The most important joints in dam site are shown in table 3 [5].

ν	φ (°)	c (MPa)	γ (ton/m^3)	m_i	σ_c (MPa)	Zone
0.288	45.81	2.16	2.45	10.6	9.14	Right Embankment
0.29	44.17	2.41	2.44	9.42	8.08	Left Embankment
0.284	46.38	2.28	2.44	12.07	9.96	Bed

Table 1: geomechanical properties of intact rocks from various zone of the dam site [5]

φ (°)	c (MPa)	a	s	m_b	E_m (MPa)	GSI	Zone
25.9	0.336	0.516	0.0012	1.28	1664.71	37.92	Right Embankment
24.69	0.31	0.517	0.00106	1.1	1474.98	37.34	Left Embankment
27.33	0.36	0.516	0.00128	1.36	1802.1	43.15	Bed

Table 2: geomechanical properties of rock masses in various zones of the dam site [5]

φ (°)	c (MPa)	Dip(°)	Dip Direction(°)	Joint Length (m)	Joint Spacing (cm)
15	0.02	77	124	3-20	20-50
15	0.02	46	291	10-20	40-60

Table 3: properties of the dominant joint sets in each zone [5]

3. Calculation of bearing capacity of the bedrock in Safa Dam

3.1. Estimation of bearing capacity using the analytical relationships

1) Determination of bearing capacity using Serrano and Olalla approach for isotropic rock masses [1]

To determine the ultimate bearing capacity, the following procedure was pursued:

- Using by values of Hock&Brown Parameters (m,s) obtained of Eq.1, values of rock mass's geomechanic parameters (β,ζ) obtaine using by Eq.2:

$$m = m_i . \exp \frac{DMR - 100}{28} \quad , \quad s = \exp \frac{DMR - 100}{9} \tag{1}$$

$$\beta = \frac{m\sigma_c}{8} \quad , \quad \zeta = \frac{8s}{m^2} \tag{2}$$

(σ_c: uniaxial compressional strength of the intact rock)

- Determination of conditions in boundary 1

To simplify the calculations, stress values were divided into β parameter to obtain normalized dimensionless parameters:

$$\sigma_1^* = \frac{\sigma}{\beta} \rightarrow \sigma_{01}^* = \sigma_1^* + \zeta \rightarrow tan i_1 = \frac{tan i_{01}}{1+\frac{\xi}{\sigma_1^*}} \quad (3)$$

(i_{01}: actual load deviation in boundary 1, i_1: the effective load deviation in boundary 1)

- Calculation of boundary 1

The instant friction in boundary 1 (ρ_1) is obtained knowing value of σ_{01} from Eq.4:

$$\sigma_{01}^* = cos i_1 \left[cos i_1 \frac{cot^2\rho_1}{2} - \frac{1-sin\rho_1}{sin\rho_1} \times \sqrt{1-\left(\frac{sin i_1}{2tan\rho_1 tan\mu}\right)^2} \right] \quad (4)$$

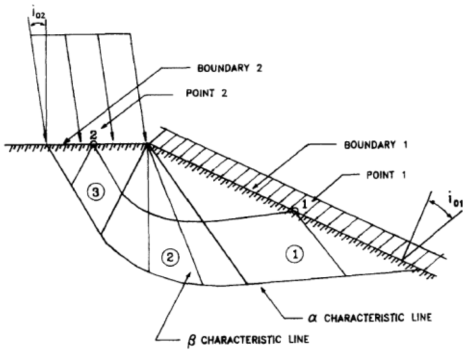


Figure 1: diagram of boundaries 1 and 2 and the network of characterizing lines under the foundation [1]

The value of ψ_1 using the Eq.5:

$$\psi_1 + \alpha = \frac{\pi}{2} - \varepsilon \quad (5)$$

Where,

ψ: the angle between the principal stress and x axis in Cartesian system,

α: the angle between boundary 1 and the horizon

$$\varepsilon = \frac{1}{2} sin^{-1} \left[cos i_1 \frac{1+sin\rho_1}{2sin\rho_1} - \sqrt{1-\left(\frac{sin i_1}{2tan\rho_1 tan\mu}\right)^2} \right] \quad (6)$$

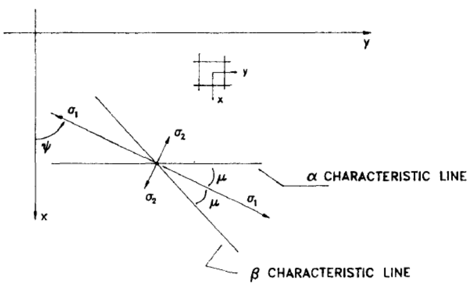


Figure 2: notification in the actual plane [1]

Once having ρ_1, Reiman constant ($I(\rho_1)$) can be determined though Eq.7:

$$I(\rho_1) = \frac{1}{2}\left[cot\rho_1 + Ln\left(cot\frac{\rho_1}{2} \right) \right] \quad (7)$$

- Calculation of boundary 2

1) Assuming an initial value for ψ_2 (for example $\psi_2 = 0$), the value of $I(\rho_2)$ can be determined through Eq.8:

$$I(\rho_2) + \psi_2 = I(\rho_1) + \psi_1 \quad (8)$$

2) Once having $I(\rho_2)$, ρ_2 can be determined using the following equation:

$$I(\rho_2) = \frac{1}{2}\left[cot\rho_2 + Ln\left(cot\frac{\rho_2}{2}\right)\right] \quad (9)$$

3) After calculation of ρ_2, the new value of ψ_2 is obtained from Eq.10. These three steps are repeated until ψ_2 limits to a constant value.

$$\psi_2 = \frac{1}{2}sin^{-1}\left[cosi_2\frac{1+sin\rho_2}{2sin\rho_2} - \sqrt{1-\left(\frac{sini_2}{2tan\rho_2 tan\mu}\right)^2}\right] \quad (10)$$

- determination of the ultimate bearing capacity

$$P_h = \beta(N_\beta - \zeta) \quad (11)$$

$$N_\beta = cosi_2\left[\frac{cot^2\rho_2}{2} + \frac{1-sin\rho_2}{sin\rho_2} \times \sqrt{1-\left(\frac{sini_2}{2tan\rho_2 tan\mu}\right)^2}\right] \quad (12)$$

- determination of factor of safety

$$F = F_p \times F_m \quad (13)$$

- Determination of Fp

The values of Fp includes uncertainties in parameters m_i, σ_c, and RMR. Generally F_p varies between 5 and 40.

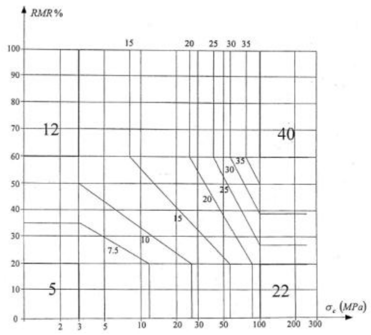


Figure 3: Proposed values relative safety factor (F_p) based on variations of rock properties [1]

- Determination of F_m

This factor of safety depends on the uncertainties in plasticity model. About highly jointed rocks and extensive foundations – which have plastic behavior – there is no need to consider F_m since $F_m = 1$. However, once the behavior is brittle this coefficient must be considered in the calculations. For intact rocks which has local or general brittle behavior, F_m depends on rock type, dimension of the foundation, and uniaxial compressional strength of the rock.

q_a (MPa)	F_m	F_p	P_h (MPa)	N_β	ζ*(10^3)	β	Zone
0.39-0.46	1	10	3.91-4.61	4.61	3.67-4.52	0.85-1	Right Embankment
0.29-0.34	1	7.5-10	2.93-3.48	4.65	4.37-5.26	0.63-0.75	Left Embankment
0.52-0.59	1	10	5.22-5.94	4.57	2.94-3.57	1.1-1.3	Bed

Table 4: the parameters value as well as the ultimate and allowed bearing capacity in Serrano and Olalla technique for isotropic rocks

2) Determination of bearing capacity for anisotropic and discontinuous rock masses [2]

In this approach, the general trend in determination of bearing capacity is based on the concept “instant friction angel” and the position and orientation of planes of weakness. The shape and strength properties of the rock is categorized into 6 groups where for each group a degradation mechanism can be assigned as follows (Figure 4):

1. MI: the overall isotropic degradation in rock mass
2. M1: degradation initiates in rock mass and under the foundation (boundary 2) and ends along weak planes in the free surface (boundary 1).
3. M2: degradation initiates along the planes of weakness under the foundation (boundary 2) and ends in the free surface of the mass rock.
4. MC: degradation initiates in rock mass and under the foundation (boundary 2), develops in planes of weakness placed in the center of disrupted surface, and ends in free surface of the rock mass (boundary 1).
5. MS: degradation is completely developed by a sliding wedge along the planes of weakness. This mechanism is called as “plane slide” in slopes stability.
6. MD: the overall disrupted mass becomes plastic and completely changes under the foundation (boundary 2). In boundary 2, degradation develops throughout the rock mass as well as in the planes of weakness.

In this technique, first failure mechanism is determined for every section of the dam site. Then, according to the failure mechanism and rock mass parameters, the coefficient of bearing capacity reduction is determined for each section.The ultimate bearing capacity is obtained by applying this coefficient on homogeneous and continuous state of bearing capacity.

- Determination of geometrical parameters

$\alpha = 0°$, $\chi = 13°$

(α:boundary deviation with horizon, χ:deviation of planes of weakness with vertical direction)

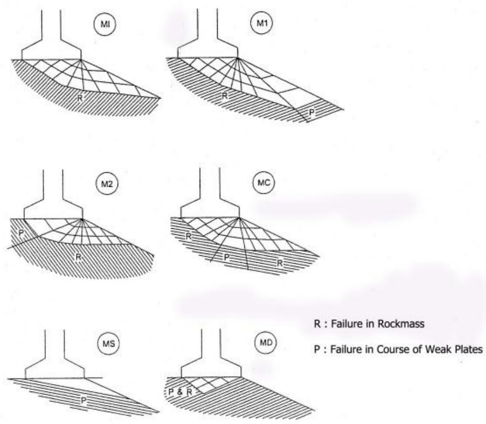


Figure 4: the potential degradation mechanisms [2]

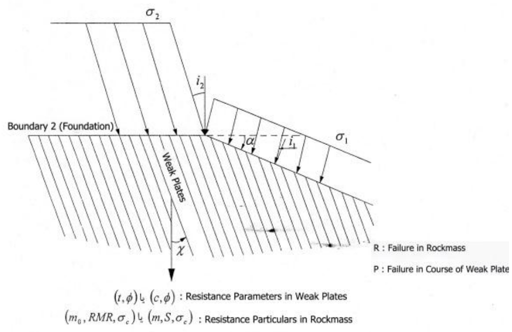


Figure 5: the created anisotropy by planes of weakness [2]

- Determination of geomechanical parameters

Determination of rock mass parameters:

$$\beta = \frac{m_0 \sigma_c}{8} exp\left(\frac{DMR-100}{28}\right) = 0.84 - 1, \zeta = \frac{8}{m_0^2} exp\left(\frac{DMR-100}{25.2}\right) = 0.00369 - 0.0045 \quad (14)$$

Determination of parameters related to planes of weakness:

c = 0.02 KPa φ = 15°

$$t = \frac{c}{\beta} cot\varphi = 0.075 - 0.09 \quad (15)$$

- Determination of boundary 1 conditions

$\sigma_1 = 0 \rightarrow \sigma_1^* = \frac{\sigma_1}{\beta} = 0 \rightarrow \sigma_{01}^* = \sigma_1^* + \zeta = \zeta = 0.00369 - 0.0045$ ، $i_1 = i_{01} = 0$

- Determination of degradation mechanism

To determine potential degradation mechanism, first the critical deviation angle in planes of weakness ($\chi_{1a}, \chi_{2a}, \chi_{1b}, \chi_{2a}$)is determined as follows:
Based on the values of σ_{01}^* and according to Eq. 4, $\rho_1 = 65.96 - 67.1$.
The values of Lamb parameters in boundary 1 are determined as:

$$q_1 = \frac{1 - sinp_1}{sinp_1} = 0.0856 - 0.095 \quad , \quad p_1 = \frac{cot^2 \rho_1}{2} - \zeta = 0.085 - 0.095 \quad (16)$$

$$\omega_1 = sin^{-1}\left(\frac{p_1^* + t^*}{q_1^*} sin\varphi\right) = 25.84 - 34.01 \quad (17)$$

$$\psi_1 + \alpha = \frac{\pi}{2} - \varepsilon_1 \quad (18)$$

$$\varepsilon_1 = \frac{1}{2} sin^{-1}\left[sini_{01}\left(cosi_{01}\frac{1+sin\rho_1}{2sin\rho_1} - \sqrt{1 - \left(\frac{sini_{01}}{2tan\rho_1 tan\mu_1}\right)^2}\right)\right] \quad (19)$$

$i_{01} = 0 \quad \rightarrow \quad \varepsilon_1 = 0 \ , \ \alpha = 0 \quad \rightarrow \quad \psi_1 = \pi/2$

$$\chi_{1a} = \psi_1 + \frac{1}{2}(\omega_1 - \phi) = 95.42 - 99.5 \quad (20)$$

$$\chi_{1b} = \psi_1 - \frac{1}{2}(\omega_1 + \phi) + \frac{\pi}{2} = 151.41 - 163.66 \quad (21)$$

In boundary 2, we have:

$$\psi_2 = \frac{1}{2} sin^{-1}\left[sini_{02}\left(cosi_{02}\frac{1+sin\rho_2}{2sin\rho_2} - \sqrt{1 - \left(\frac{sini_{02}}{2tan\rho_2 tan\mu_2}\right)^2}\right)\right] \quad (22)$$

$i_{02} = 0 \rightarrow \psi_2 = 0$

Based on the values of ρ_1, ρ_2 is determined according to the technique explained in continuous approach. The values of and can be determined likewise boundary 1. Once having these 4 angles, degradation mode can be easily determined. For instance, for right abutment – where X = 13, we have:

Failure mode = M2

- Determination of bearing capacity

For instance, based on rock mass properties and its failure mode and using the related diagrams interpolation for the right abutment we have:

Coefficient of bearing capacity reduction: 0.92

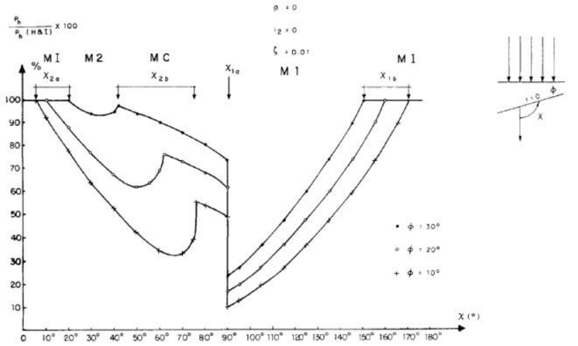


Figure 6: coefficient of bearing capacity reduction induced by the anisotropy for ($\zeta = 0.01$, $i_2 = 0$, $\sigma_1 = 0$, $t = c = 0$, and $\alpha = 0$) [3]

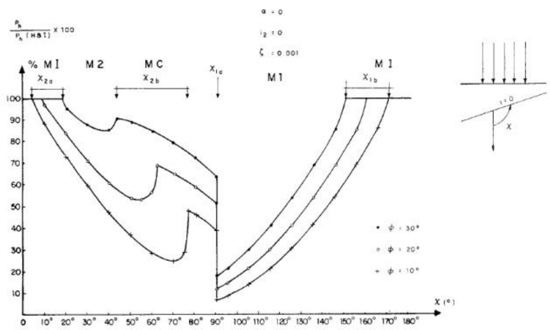


Figure 7: coefficient of bearing capacity reduction induced by the anisotropy for ($\zeta = 0.001$, $i_2 = 0$, $\sigma_1 = 0$, $t = c = 0$, and $\alpha = 0$) [3]

q_a (MPa)	P_h (MPa)	Coefficient of bearing capacity reduction	Failure mode	Zone
0.37-0.43	3.71-4.38	0.92	M2	Right Embankment
0.29-0.34	2.93-3.48	1	M2	Left Embankment
0.31-0.35	3.1-3.56	0.52	M1	Bed

Table 5: the values of the parameters and the ultimate bearing capacity through Serrano and Olalla technique for anisotropic rock masses

3.2. Determination of bearing capacity by numerical approaches

1- Finite element model of the dam and rock site

For the numerical analyses, the applied software was ABAQUS through which a simulated environment known as jointed material is utilized to model the jointed rock. In this model, the joint sets were simulated as weak close planes. In this state, the analysis is conducted as elastoplastic mode. To mesh to model, pyramidal, tetragonal, 4-node elements were used.

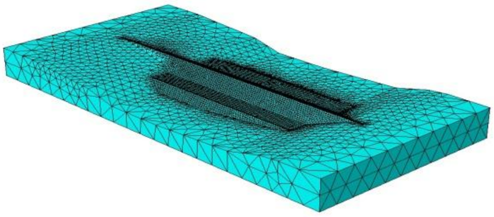
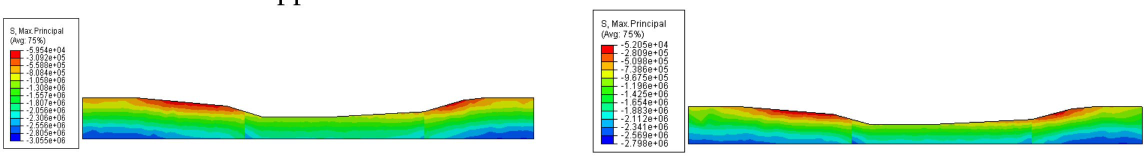


Figure 8: the meshed model of the foundation and dam body

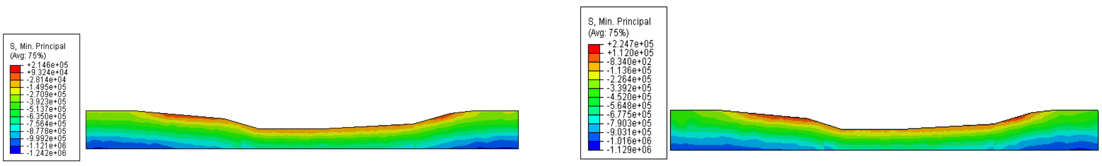
2- Numerical analysis of the rock in foundation and abutments

To investigate the effect of modeling of the joint sets in rock mass on induced stresses, stress analysis was performed in two states. In state 1, the bedrock was modeled as continuous without considering the joint sets and applying rock mass parameters; while in state 2, intact rock parameters were assigned to the bedrock through modeling of the two joint sets introduced in Table 3. Here, the compressional and tensional stresses were notified as negative and positive sings, respectively. All stress values are in MPa. The load induced by bedrock weight and dam body was applied as weighted. Also, the water mass behind the dam was applied to the model.



a) b)

Figure 9: contour of the major principal stresses: a) without considering joint sets, b) considering the joint sets



a) b)

Figure 10: contour of the minor principal stresses: a) without considering joint sets, b) considering joint sets

3- Estimation of safety factors and the allowed bearing capacity

To determine safety factors, degradation criterion of Hoek and Brown (Eq. 23) is applied.

$$\sigma_1 - \sigma_3 = \sigma_{ci}\left(m_b \frac{\sigma_3}{\sigma_{ci}} + s\right)^a \tag{23}$$

In Eq. 23, degradation factor of the difference between the major and minor principal stresses was determined from the software (left side of the equation) while the resisting factor is the right side of the equation. Safety factors were determined for abutments and bedrock in Safa Dam and the results were shown in Table 6.

To determine the allowed bearing capacity, first the overall compressional resistance of the rock mass is determined from Eq. 24, proposed by Hoek and Brown, and then it is divided into the safety factor.

$$\sigma'_{cm} = \sigma_{ci} \frac{[m_b + 4s - a(m_b - 8s)] \times \left(\frac{m_b}{4} + s\right)^{a-1}}{2 \times (1+a) \times (2+a)} \tag{24}$$

The results of these calculations are shown in Table 10.

Allowed Bearing Capacity (MPa)		Safety Factor		Compressional Resistance of The Rockmass (MPa)	
discontinuous	continuous	discontinuous	continuous		
0.33-0.51	0.38-0.54	3.1-4.72	2.9-4.1	1.59	Right Embankment
0.3-0.49	0.33-0.5	2.58-4.24	2.52-3.82	1.27	Left Embankment
0.58-0.76	0.66-0.88	2.3-3	2-2.65	1.76	Bed

Table 6: Values of safety factors and allowed bearing capacity in both states in numerical method

Bed	Left Embankment	Right Embankment	
0.52-0.59	0.29-0.34	0.39-0.46	Serrano&Olalla Method for Continuous State
0.32-0.35	0.29-0.34	0.37-0.44	Serrano&Olalla Method for Discontinuous State
0.66-0.88	0.33-0.5	0.38-0.54	Numerical Method for Continuous State
0.58-0.76	0.3-0.49	0.33-0.51	Numerical Method for Discontinuous State

Table 7: Comparison of values of allowed baring capacity in both methods

4. Summation and conclusion

1) A comparison between bearing capacity values obtained from Serrano and Olalla approaches and numerical technique in both jointed and non-jointed states indicates that presence of joint sets and planes of weakness can significantly affect bearing capacity of the bedrock. This, in turn, depends on joints orientation and spacing as well as joints and infillings conditions; thus, the accurate conditions and strength of the joints present in rock environments are of high significance.

2) Serrano and Olalla technique is in proper agreement with numerical method and the average of the allowed bearing capacity for the abutments and foundation in both methods are close them.

3) Serrano and Olalla technique in anisotropic and discontinuous state, because of accurate survey conditions of joints, is best method for determination of bedrock bearing capacity.

5. References

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