A Comparative Review with respect to Estimating the Bearing Capacity of Shallow Foundations on Jointed Rock Masses in Hoek-Brawn and Non Hoek-Brawn Rocks

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Abstract: One of the main issues regarding the design of foundations is how to determine their bearing capacity. So far, the bearing capacity of the foundations constructed on various soils has widely been studied, yet few studies have been conducted concerning the bearing capacity of rock foundations. However, in many civil projects across the world, structures foundation is constructed on joined rock masses. The bearing capacity of such rock masses are mostly estimated either using quasi-empirical equations or based on local codes. Amongst the studies carried out on the bearing capacity of the joined rock masses, still no comprehensive applied studies has not been considered in executive works for such rocks. In the present article, different methods are collected with regard to the estimation of the bearing capacity of shallow foundations built on jointed rock masses by conducting comprehensive literature reviews, and the hypotheses and equations related to each one are examined in applied terms. Accordingly, first, the whole methods were classified in two groups (i.e. Hoek-Brown & Non-Hoek-Brown) after dividing all existing equations, and the formulation of each method was generalized in the same way so that a logical framework could be gained for comparing between different methods and determining the optimum one in the estimation of the bearing capacity of foundations on joined rock masses. In the end, employing the generalized new equations gained, the bearing capacity of rock masses with known characteristics was calculated for a certain example using various methods and results were compared.

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1. Introduction

Estimating the bearing capacity of soil or the rock base of structure in the place of constructing the structure is among the basic steps of design, construction and execution of different structures including the foundations. So far, many studies are carried our regarding the bearing capacity of soils. And, a variety of theoretical and/or numerical models are presented with respect to the strength parameters of soil for determining the bearing capacity of the foundations built on different soils [1]. Yet, due to their highly varying nature and the great effect of physical and geometric characteristics of the joints on bearing capacity, it is not possible to present an explicit solution for calculating the bearing capacity of such foundations (i.e. rock foundations) in rocks. As compared to soils; being filled by water, minerals, and/or other existing characteristics can be implied among the properties of joints [1]. In the present article, a general framework is developed based on different methods of estimating the bearing capacity of shallow foundations built on jointed rock masses by conducting comprehensive literature reviews, and examining the conditions of joints. Accordingly, first, the whole methods were classified in two groups (i.e. Hoek-Brown & Non-Hoek-Brown) based on the distribution status of joints and also their characteristics, and then the estate of estimating the bearing capacity was presented for each group. That is to say, the formulation of each method was generalized in the same way so that a logical framework could be gained for comparing between different methods and determining the optimum one in the estimation of the bearing capacity of foundations on joined rock masses.

Generally, based on the classification done here, by "Hoek-Brown rocks" we mean the rocks their failure complies with Hoek-Brawn criterion [2]. These rock masses include either intact rocks with no joints or jointed rocks with near joints and blocks composed of such joints are considered as very small compared to the dimensions of the structure. In addition, in the first group (i.e. Hoek-Brawn rocks), all joints have similar surface properties and none of them is weaker than the others. Upon classifying the rocks in the first group mentioned above, the rock mass will be considered as a non-Hoek-Brawn rock mass and will undergo different analyses, provided that one of the conditions described for Hoek-Brawn rock is not met.

2. The Bearing Capacity of Hoek-Brawn Rocks

In general, two methods are applied for determining the bearing capacity of Hoek-Brawn rock masses including limit equilibrium method and characteristic lines method. In the former, the bearing capacity is calculated by presuming a failure mechanism and employing Hoek-Brawn failure equation. Yet, in the latter, first the differential equations dominant over a rock circumference are set with respect to the stress characteristics and applying Hoek-Brawn failure equation, and then the bearing capacity beneath the foundation is gained by employing boundary conditions near the foundation and solving the differential equations using finite difference methods. In the forthcoming, respective general formulae are presented.

2.1. The Bearing Capacity of Hoek-Brawn Rocks based on Limit Equilibrium Method

To determine the bearing capacity of Hoek-Brawn rock masses using limit equilibrium method, first, two zones are considered in beneath the foundation: active zone and passive zone. Then, taking the direction of normal or original stresses to rock surface and employing the equilibrium on the mass circumference, the final amount of bearing capacity of foundation will be determined in various depths of foundation in rock based on following equations [4].

(1) $$q_{ult} = C_{f1}\sigma_C\sqrt{S}\left[1+\sqrt{(m/\sqrt{S})+1}\right] \quad for \quad D_f = 0$$

(2) $$q_{ult} = C_{f1}\left[\sqrt{m\sigma_c\sigma_3 + S\sigma_c^2} + \sigma_3\right] \quad for \quad D_f \neq 0$$

In Equations (1) and (2), parameter Df stands for the buried depth of foundation in rock, σ_c for uniaxial compressive strength of intact rock, and C_{f1} for the correction coefficient of foundation shape considered to be 1.2 and 1.25 respectively for circular and rectangular footings and 1 for strip footing. In addition, parameter q is the rock mass load and parameters m and S indicate Hoek and Brawn parameters. Also, they equal the main stress in horizontal direction and will be determined using Equation (3).

(3) $$\sigma_3 = \sqrt{m\sigma_c q + S\sigma_c^2} + q$$

2.2. The Bearing Capacity of Hoek-Brawn Rocks based on Joint Characteristic Lines Method

In 1994 and 1996, Serrano and Olalla [6,7] developed a more comprehensive method for determining the bearing capacity of rock foundations using Hoek-Brawn failure criterion and applying the characteristic line method. After resetting the presented equations in their method, the final bearing capacity of rock foundation is gained via the following equation:

(4) $$q_{ult} = \beta(N_\beta - \lambda)$$

The above equation will gain the final bearing capacity of rock foundation based on N_β, β, and v. The formula of each of the parameters is presented in equations (5) to (7):

(5)
$$N_\beta = \cos(i)[0.5\cos(i).\cot^2(\rho) + \frac{1-\sin(\rho)}{\sin(\rho)}\sqrt{1-(\frac{\sin(i)}{2\tan(\rho).\tan(\pi/2-\rho/2)})^2}]$$

(6) $$\beta = \frac{m\sigma_c}{8}$$

(7) $$\lambda = \frac{8S}{m^2}$$

In the above equations, parameters m and S indicate Hoek and Brawn parameters, as mentioned above. And, i stands for bearing angle beneath foundation to vertical direction, σ_c for uniaxial compressive strength of intact rock, and ρ for instantaneous friction angle under ultimate load. For boundary conditions akin to what is illustrated in Figure (1), it is possible to gain the value of ρ from solving Equation (8) based on ρ and q is the load on the foundation in this equation.

$$\text{cotan}(\rho) + Ln(\text{cotan}(\frac{\rho}{2})) = \text{cotan}(\sin^{-1}(\frac{1}{1+(2(\frac{q}{\beta}+\lambda))^{0.5}})) + Ln(\text{cotan}(0.5\sin^{-1}(\frac{1}{1+(2(\frac{q}{\beta}+\lambda))^{0.5}}))) + \pi - 4$$
(8)

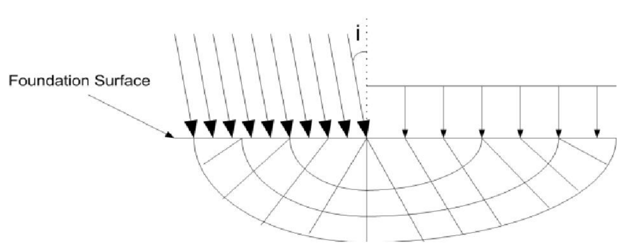

Fig. 1- Boundary conditions employed on the characteristic lines network beneath foundation

2.3. The Bearing Capacity of Hoek-Brawn Rocks based on Characteristic Lines Method and the Modified Version of Hoek-Brawn Criteria

In the method implied in section (2.2), the basic version of Hoek-Brawn failure criterion is used for determining the bearing capacity of the rock masses. But applying the criterion leads us to not to have a good estimation of severely broken rocks or highly jointed rocks. Hence, Serrano and Olalla [8] improved their method using the modified version of Hoek-Brawn criterion. Then, by introducing new parameter *n* the value which varying between 0.5 and 0.65, it is possible to include the excessive breakage of the rock mass in determining the bearing capacity of rock masses. Accordingly, upon resetting the equations above, the bearing capacity of the rock mass is estimated by the modified version of Hoek-Brawn criterion using Equation (4); except the matter that parameters N_β, β, and λ in this method are similar the equations presented in section (2.2) and gained based on Equations (9) to (11).

All parameters used in equations below are similar to the parameters presented in section (2.2).

$$(9)\quad N_\beta = \cos(i).\left(\frac{1-\sin(\rho)}{(\frac{1-n}{n})\sin(\rho)}\right)^{(n/1-n)}.[n(\frac{1+\sin(\rho)}{\sin(\rho)}).\cos(i)+\sqrt{1-[n(\frac{1+(\frac{1-n}{n})\sin(\rho)}{\sin(\rho)}).\sin(i)]^2}]$$

$$(10)\quad \beta = \sigma_c.\left(\frac{m(1-n)}{2^{(1/n)}}\right)^{(n/1-n)}$$

$$(11)\quad \lambda = \frac{S}{m(\frac{m(1-n)}{2^{(1/n)}})^{(n/1-n)}}$$

2.3. The Bearing Capacity of Hoek-Brawn Rocks based on the Mid Main Effective Stress

In the methods presented in previous sections, the effect of the mid main effective stress for the bearing capacity is ignored in estimating Hoek-Brawn rocks bearing capacity. But, observing the results from triaxial tests done on rock masses, Zhou et al [9] found out that the bearing capacity increases when the mid main effective stress is included in calculations. As a result, employing unified failure criterion [10] and characteristic line method, they have calculated the bearing capacity of the Hoek-Brawn rocks. Resetting the equation of bearing capacity based on Equation (4), the indices of bearing capacity of previous sections are determined based on Equations (12) to (15) with respect to the mid stress effect.

In the equations below, parameter GSI is the index of geological conditions introduced in the modified version of Hoek-Brawn criterion. Other parameters are like the previous sections.

$$(12)\quad N_\beta = \frac{\alpha}{1+b}.\cos^2(i).(\frac{(1-\alpha/\alpha).\sin\rho+1+b}{\sin\rho})(\frac{1+b-\sin\rho}{(1-\alpha/\alpha).\sin\rho})^{\alpha/1-\alpha}+\cos(i).(\frac{1+b-\sin\rho}{(1-\alpha/\alpha).\sin\rho})^{\alpha/1-\alpha}.\sqrt{1-[\frac{\alpha}{1+b}.(\frac{(1-\alpha/\alpha).\sin\rho+1+b}{\sin\rho}).\sin(i)]^2}$$

$$(13)\quad \beta = \sigma_c(1+b).[\frac{m(1-\alpha)}{(2+b)^{1/\alpha}}]^{\alpha/1-\alpha}$$

$$(14)\quad \lambda = \frac{S}{m.(1+b).[\frac{m(1-\alpha)}{(2+b)^{(1/\alpha)}}]^{\alpha/1-\alpha}}$$

$$(15)\quad \alpha = \frac{1}{2}+\frac{1}{6}[\mathrm{Exp}(-\frac{GSI}{15})-\mathrm{Exp}(-\frac{20}{3})]$$

3. The Bearing Capacity of Non Hoek-Brawn Rocks

As also mentioned earlier, Non-Hoek-Brawn rocks are rock masses with either an infinite number of joints or with their building blocks much larger than the dimensions of the structure and/or show anisotropic behavior due to many joints in them. As a result, three general modes are considered for determining the bearing capacity of such rocks which are presented in the next sections.

3.1. Rock Masses with a Set of Joints

In some cases, the foundation may be located on the rock mass with only one set of joints. Then, if we suppose the foundation loading vertically, three modes can be assumed for the shape of joints arrangement [3] each of which will be introduced in the following sections and the bearing capacity will be determined in such rock masses.

3.1.1. The Set of Horizontal Joints or Joints with very small Slope to Horizon

If the joints existing in a rock mass has a slope less than 20° to horizon and the material of rock is not changed between the strata formed by joints, it is possible to calculate the bearing capacity of that rock mass based on the bearing capacity of intact rock implied in sections related to Hoek-Brawn (2.2 to 2.4.). Here, two special sections are examined:

a) **When a Soft Clay Layer appeared between the Layering done by Joints (Fig. 2)**

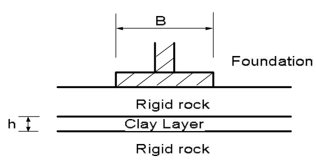

Fig. 2. the clay layer between rock strata

Now, the loading capacity of the rock mass is set based on Equation (16).

$$(16)\quad q_{ult} = S_u B / 2h$$

Where q_{ult} is the ultimate bearing capacity of the rock mass and determined based on parameters B (foundation width), h (clay layer thickness), and S_n (the shear strength of non-drained clay). It must be noted that in this method the clay layer must be located in the area of stress bubble beneath the foundation.

b) When the Soft Rock with Great High beneath the Rigid Rock with Small High (Fig. 3)

Now, first, the multiplication of circumference to the rigid layer high is determined and then the value gained is multiplied to the shear strength of the rigid intact rock. Also, the strain strength of the rigid intact rock is set and afterward the smaller value is considered as the bearing capacity of the rock mass. It is noteworthy that the philosophy underlying how to choose the least or minimum value between the values mentioned is: the dominance of penetrative and/or bending failure mechanism in the rock mass failure.

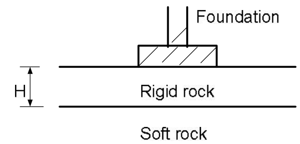

Fig. 3- the soft rock beneath rigid rock layer

3.1.2. The Set of Oblique Joints

When the set of joints forms a slope between 20° and 70° to horizon (Figure 4), the bearing capacity of the rock mass can be calculated with respect to general failure mechanism based on Equation (17).

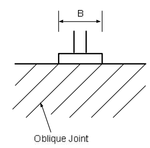

Fig. 4- the set of oblique joints

$$(17)\quad q_{ult} = 0.5.(\tan^5\alpha - \tan\alpha).B\gamma + 0.85q.\tan^4\alpha$$

As before, here, q_{ult} is the ultimate bearing capacity of the rock mass, B foundation width, γ the special weight of the rock mass, q the load existing on the foundation, and α equals $45 + (\phi/4)$ and determined based on ϕ (the inner friction angle of the rock mass) by Mohr-Columb criterion.

3.1.3. The Set of Vertical or Near Vertical Joints

In cases where the set of joints are vertical with an angle between 70° and 90° to horizon (Figure 5), two following modes can be imagined based on the distance between joints and the dimensions of the foundation:

a) The Distance between Joints is more than Width (S>B)

If the joints are open, then the failure mode of the rock mass is in form of compressive failure of separate rock pillars and its bearing capacity is gained based on Equation (18).

$$(18)\quad q_{ult} = 2C.\tan\alpha$$

Where parameter C stands for the viscosity of the rock mass and gained based on Mohr-Columb criterion.
If joints are closed, the failure mode of the rock mass is

in form of general shear failure and the bearing capacity in soil can be set for this by equating the behavior of rock based on Mohr-Columb criterion and based on the same equations.

b) The Distance between Joints is less than Width (S<B)

Now, the bearing capacity of the foundation is gained based on equation (19).

(19)

$$q_{ult} = \eta.[((2JS/B).\sin^2\alpha.\cot\phi).(\tan^2\alpha - 1) - (\tan^2\alpha.\cot\phi) + 2C.\tan\alpha]$$

In Equation (19), parameters C and ϕ respectively stand for viscosity and the friction angle of the rock mass based on Mohr-Columb criterion. Also, parameter S is the distance between the joints, B foundation width, η the correction index related to the shape of foundation and J the correction index related to the height of the rock layer. The value of η is respectively 1 and 0.85 for circular and square footings and $1/(2.2+0.18(L/B))$ for strip footing where L is the length of foundation. To determine J, Equation (20) is applied where H is the height of the rock layer.

(20)

$$J = \begin{cases} 0.42 & for\ \frac{H}{B} \le 0.5 \\ -0.0113(\frac{H}{B})^2 + 0.1702(\frac{H}{B}) + 0.3696 & for\ 0.5 < \frac{H}{B} < 7 \\ 1 & for\ \frac{H}{B} \ge 7 \end{cases}$$

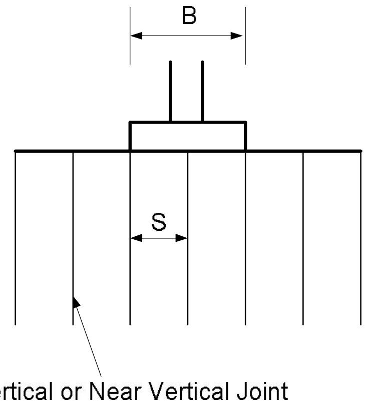

Fig. 5. the set of vertical or near vertical joints

3.2. The Rock Masses with a Limited Set of Joints

If the rock mass has a limited number of joint set and they form one or more failure wedge dimensions, one of the joint sets will spontaneously play the failure wedge beneath the foundation (Figure 6). On the other hand, failure wedge is reduced in parallel with the joints surfaces and the bearing capacity of the rock is declined severely. This is because, in this mode, the shear strength of the joints is significantly less than the strength of the rock mass and the dimensions and the surface of the failure wedge is also limited. In such rocks, the bearing capacity can be calculated via Equation (21) by considering two sets of vertical joints with slopes ψ_1 and ψ_2 to horizon [4].

$$q_{ult} = [(q + \frac{\gamma B}{2\tan\psi_1}).\tan^2(45+\frac{\phi_2}{2}) + (\tan^2(45+\frac{\phi_2}{2}) - 1).\frac{C_2}{\tan\phi_2}].\tan^2(45+\frac{\phi_1}{2}) + (\tan(45+\frac{\phi_1}{2}) - 1).\frac{C_1}{\tan\phi_1}$$

(21)

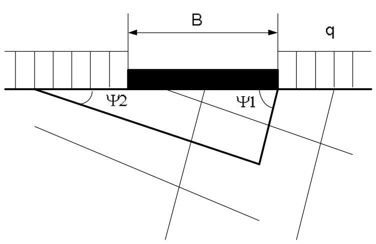

Fig. 6. the formation of failure wedge beneath foundation by joints

As also introduced earlier, parameters C and ϕ respectively stand for viscosity and the friction angle of the rock mass based on Mohr-Columb criterion. Also, it must be noted that the indices presented indicate the number of each of the joint sets. In this method, the corresponding bearing capacity for both joint sets is gained by placing the parameters related to each of the equation above and then minimum bearing capacity gained is taken as design criterion (i.e. the bearing capacity of the rock mass).

3.3. The Rock Masses with many Unharmonious and Anisotropic Joints

Using Bell solution and considering two active and passive zones beneath the foundation as well as applying the parabolic strength criterion (Figure 5), Singh and Rao [2] have presented the bearing capacity of anisotropic jointed rocks. Based on the method, the area beneath the foundation (Figure 7) is divided into two parts. Here, the value of parameter J_f is determined for all sets of joints existing in areas (I) and (II). This parameter depends on the frequency, orientation, and strength of the joints and the failure mode of the rock mass and is gained based on Equation (22).

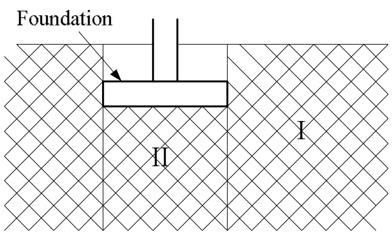

Fig. 7- the zoning of the area beneath foundation

(22) $J_f = \dfrac{J_n}{n.\tan(\phi_j)}$

Where J_n stands for the number of joints in 1m length of loading, ϕ_j for the inner friction angle of the joint, and n for an empirical parameter which depends on the angle between normal direction and loading direction (Θ) with the plane set based on Table (1).

Table 1- the values of a and n parameters based on different θ values

Θ(°)	0	10	20	30	40	50	60	70	80	90
n	1	0/814	0/634	0/456	0/306	0/071	0/046	0/105	0/460	0/810
a	-0/0123	-0/0123	-0/018	-0/018	-0/018	-0/018	-0/018	-0/018	-0/025	-0/0123

After determining J_f, the value of σ_{cj} is set for each set of joints via the following equation.

(23) $\sigma_{cj} = \sigma_{ci} . \mathrm{E}\,\mathrm{xp}(aJ_f)$

Where σ_{ci} is the unlimited compressive strength of the intact rock and a is the empirical parameter depending on the failure mode and being set by Table (1). After determining σ_{cj} for all sets of joints located in areas (I) and (II), their minimum values will be set separately for each of the areas. The minimum value of σ_{cj} is respectively Ψ_1 and Ψ_2 for areas (I) and (II). Then, the ultimate bearing capacity of the foundation will be calculated based on Equations (24) to (26).

(24) $q_{ult} = A(\sigma_1)^2 + (1.2A.\sigma_{ci}).\sigma_1 + \Psi_2$

(25) $\sigma_1 = A.(\gamma_r D_f)^2 + (1.2A\sigma_{ci}).\gamma_r D_f + \Psi_1$

(26) $A = -1.23(\sigma_{ci})^{-0.77}$

In the equations, parameters D_f and γ_r respectively stand for the buried depth of the foundation and special weight of the rock.

4. Presenting A Numerical Example to Specify the Optimized Method for Determine the Bearing Capacity of Shallow Foundations

Regarding the fact that it is not possible to develop a general rule with respect to the non-Hoek-Brawn rocks and each of the equations presented in Section 3 can be used depending on the conditions of the problem, hence the example presented in this section is related to Hoek-Brawn rocks, per se. It must be noted that, in general, with regard to the conservativeness of setting the bearing capacity of the rock foundations, it is possible to gain reliable results based on Hoek-Brawn criterion. As a result, the equations gained in this method can also be generalized to non-Hoek-Brawn rocks. To compare between different methods of determining the loading capacity of Hoek-Brawn rocks and to find the optimized method for executive problems, here a strip footing with buried depth of 2m is considered on the jointed rock mass. Theoretically, parameter PMR of the rock mass is 65, the uniaxial compressive strength of the rock is $20MN/m^2$, parameter m_0 is related to the intact rock which is 7 based on Hoek-Brawn results and the special weight of the rock mass is $26KN/m^2$. Using the characteristic method, the value of instantaneous friction angle beneath the foundation surface is gained 20°. Finally, the values related to the ultimate bearing capacity of the foundation on the respective rock mass is considered based on various methods using MATLAB programming and results are listed in Table 2.

Table 2- Analysis results for a variety of methods determining the Bearing Capacity of Hoek-Brawn Rocks

Method	β	λ	N_β	$p_h(MN/m^2)$
Limit Equilibrium Method	NA	NA	NA	6/255
Joint Characteristic Lines Method with Hoek-Brawn Criterion	1/552	0/0913	5/698	8/702
Joint Characteristic Lines Method with Modified Hoek-Brawn Criterion	0/436	0/7263	16/44	7/02
Joint Characteristic Lines Method with respect to the effect of mid stress	1/361	0/104	11/008	12/84

Based on the results from numerical analysis observed in the above Table, it is seen that the bearing capacity of the rock mass is more conservative in limit equilibrium method compared to the other methods. Also, concerning the effect of mid stress in characteristic lines method, the bearing capacity of the Hoek-Brawn rock mass is determined more compared to the other methods. On one hand, it can be said that lack of attention to the mid stress is perhaps the reason underlying the excessive conservative estimation of the bearing capacity. On the other hand, the equations gained by the characteristic method based on Mohr-Columb criterion gains close numbers whether for the modified version or the non-modified one, and shows the insignificant difference between two methods in determining the bearing capacity of the rock foundations. Moreover, it can be concluded that the bearing capacity gained based on the modified method is smaller and more or less employs the failure of the rock mass in calculating the bearing capacity.

5. Conclusion

In the present article, a variety of methods for determining the bearing capacity of the jointed rock masses were examined via conducting a comprehensive literature review regarding the bearing capacity of rock foundations. In the same vein, considering the status of joints distribution as well as their properties, rocks were classified in two groups (i.e. Hoek-Brown & Non-Hoek-Brown) and for each the estate of setting the bearing capacity was presented. In doing so, it was attempted to take their applied aspect into account so that they can be used for design purposes and the optimized method of determining the bearing capacity of the rock foundations can be introduced. Accordingly, the existing equations and formulae were turned into a general uniform framework and, in the end, the bearing capacity of Hoek-Brown rock was calculated for a special example and the results were compared. Based on the results, it is observed that the bearing capacity of the Hoek-Brown rock mass was estimated as a small amount and in a conservative way using limit equilibrium method. On a contrary, considering the mid stress led to the considerable increase of the rock bearing capacity. Results of the articles suggest that, for design purposes and regarding the status of the rock mass and the dimensions of the foundation as well as the conditions of the joints, the rock mass can be included in one of the cases discussed here and then the bearing capacity of the rock mass can be calculated using the equations presented.

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