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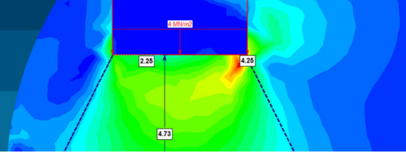

The design of shallow foundations on fractured rock

Diseño de fundaciones superficiales en roca fracturada

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Designing shallow foundations on fractured rock is a complex challenge for civil engineers due to varied geological structures and material properties. Unlike soils, estimating rock mass bearing capacity using soil mechanics methods is often unsuitable due to irregular block dimensions. Factors like discontinuities, filling materials, and fracture intensity further complicate developing a universal theory akin to Terzaghi's for soils. Peck introduced a method correlating bearing capacity with Rock Quality Designation (RQD), followed by approaches based on Bieniawski's Geomechanics Classification System and empirical methods. Despite advancements in numerical modelling, no universal solutions exist. This study analyzes fractured rock behaviour using Chilean site samples, employing finite element models to compute shear stresses and deformations. The goal is to propose a comparative method integrating empirical and numerical approaches, evaluating result dispersion.

Keywords: shallow foundations, bearing capacity, rock mechanics, finite element method

El diseño de fundaciones superficiales sobre roca fracturada representa un desafío complejo para los ingenieros civiles debido a la diversidad de estructuras geológicas y propiedades de los materiales. A diferencia de los suelos, la estimación de la capacidad portante del macizo rocoso mediante métodos de mecánica de suelos suele ser inadecuada debido a las dimensiones irregulares de los bloques. Factores como las discontinuidades, los materiales de relleno y la intensidad de la fractura complican aún más el desarrollo de una teoría universal similar a la de Terzaghi para suelos. Peck introdujo un método que correlaciona la capacidad portante con la designación de calidad de roca (RQD), seguido de enfoques basados en el Sistema de Clasificación Geomecánica de Bieniawski y métodos empíricos. A pesar de los avances en modelación numérica, no existen soluciones universales. Este estudio analiza el comportamiento de la roca fracturada utilizando muestras de sitios chilenos, empleando modelos de elementos finitos para calcular los esfuerzos de corte y las deformaciones. El objetivo es proponer un método comparativo que integre enfoques empíricos y numéricos, evaluando la dispersión de los resultados.

Palabras clave: fundaciones superficiales, capacidad de soporte, mecánica de rocas, método de elementos finitos

Introduction

Designing shallow foundations on fractured rock masses is a challenging task. The methods used for footing design on rock must consider both the intact rock properties and the characteristics of discontinuities. The complexity of geological features, such as the orientation and condition of discontinuities, weathering profiles, and construction blasting damage, increases the uncertainty of engineering designs.

Traditionally, estimating the ultimate bearing capacity of shallow foundations has relied on previous experience, empirical criteria, or national code design procedures (Serrano and Olalla, 1994). Small–scale projects may lack the extensive field and laboratory testing required for rock engineering design (Rose, 2004). Design engineers must often select strength and deformation parameters from technical literature or use a presumptive allowable bearing pressure, which may not always be conservative, depending on the site's rock conditions. Various authors have presented and extended classic rock mechanics concepts and design procedures (*e.g.*, Goodman, 1989; Wyllie, 2003; Feng and Hudson, 2011; Wittke, 2014; Aydan, 2017; Hoek, 2023). Other design procedures are found in publications by the American Society of Civil Engineers (ASCE), the American Association of State Highway and Transportation Officials (AASHTO), the Canadian Foundation Manual (CFM), and international codes for different rock foundation projects. These documents often include empirical formulas and tables intended for use by experienced engineers with a rock mechanics background, which may not be familiar to geotechnical engineers more experienced in soil mechanics.

The mode of failure (as shown in Figure 1) is influenced by the joint spacing relative to the footing size and the combination of hard and weak layers (Sowers and Sowers, 1979). The failure mode depends on whether the joints are open, closed, or wide, their orientation (vertical to horizontal), or if there is a thin rigid layer over a weak compressible layer.

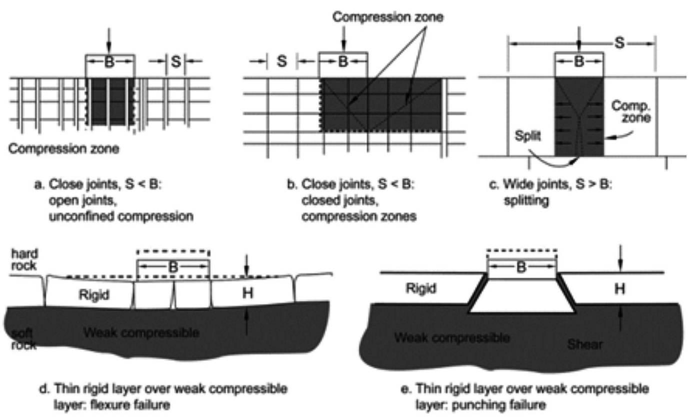


Figure 1: Bearing capacity failures modes (Sowers and Sowers, 1979)

A simplified representation of foundations transitioning from intact to heavily jointed rock mass with increasing sample size is presented by Serrano and Olalla (1996) (Figure 2), which is a modified scheme based on the idealized diagram by Hoek (1983). This representation illustrates the influence of scale on the rock mass behaviour model, which should be used in designing shallow foundations on horizontal or inclined rock masses.

A particular consideration regarding the applicability of the procedure from Serrano and Olalla (1994) and others, is to consider the Group I (intact rock) and Group IV–V (jointed rock mass) with rock isotropy and homogeneity. For complex scenarios like Group III–IV other considerations and more advanced design must be carried out.

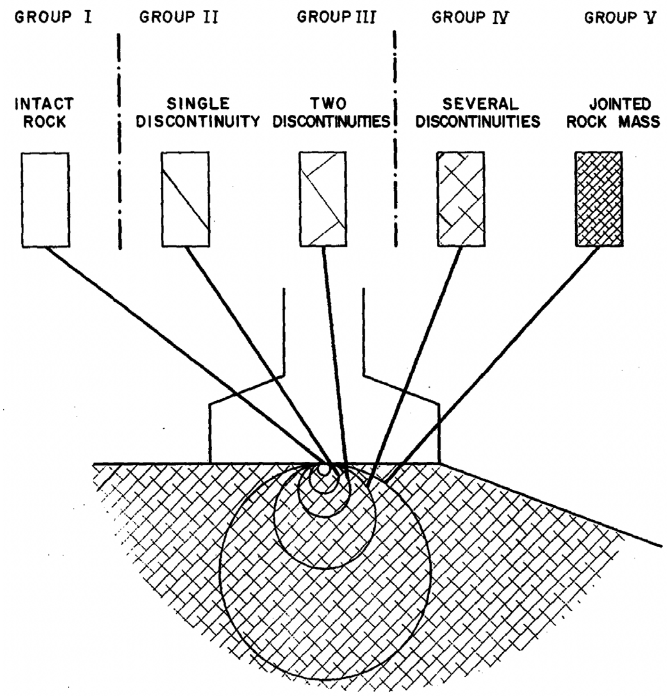
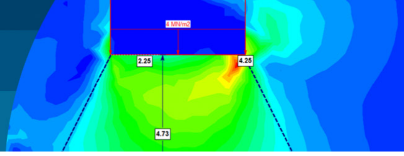


Figure 2: Simplified representation of the influence of scale on type of rock mass behaviour model which should be used in designing shallow foundations on rock slope (Serrano and Olalla, 1996).

Another feature of spread footings on rock is that the bearing surface does not have to be perpendicular to the direction of the applied load. This is because igneous rock generally has high shear strength, and if necessary, anchors can be installed to provide additional shear resistance as required by the project. Under these conditions, vertical loads can be supported on sloping rock faces, inclined loads on horizontal surfaces, and vertical loads on two levels without any issues. External loads such as wind and seismic forces may act on the structure, creating overturning moments and uplift forces. The foundation design must accommodate these conditions. If the combination of seismic and wind forces generates uplift forces, it may be necessary to design tie–down anchors to stabilize the entire system.

Most foundations on rock are spread or continuous footings at the ground surface, but there are situations where this type of footing is not feasible. This could be because the



available bearing capacity does not meet design criteria, the bearing capacity occurs at a considerable depth, or the project specifies a certain depth. In summary, the design of surface foundations on fractured igneous rock must consider the following to ensure good foundation performance: a) the ultimate bearing capacity of the fractured rock to ensure there will be no further fracturing, crushing, or creep within the loaded zone (pressure bulb); b) the maximum settlement of the foundation, which can result from a combination of elastic and plastic strain of the rock mass, as well as potential compression of weak seams within the volume of rock mass compressed by the applied load and c) sliding and shear failure and shear failure of rock blocks formed by intersecting discontinuities within the foundation's influence area. This condition typically occurs when the foundation is located on a steep slope and the orientation of the discontinuities allows blocks to slide out of the open face, or when two foundations are located too close together at different levels (as can occur with ring foundations).

A non–written recommendation is that the performance of an important foundation must be checked with respect to all of these three conditions because they are independent of each other.

Geotechnical design approaches on rock mass

Foundations on faulted rock masses can present significant challenges for the foundation engineer due to the greater heterogeneity of rock compared to soil. Spread footings supported on rock must be designed to handle the design loads with adequate bearing capacity, structural integrity, and tolerable settlements in accordance with the project requirements.

The response of footings subjected to seismic and dynamic loading should be evaluated based on local norms and experience. For footings on rock, the location of the resultant pressure (R) at the base of the footing should be kept within $B/4$ of the center of the footing of width B. The bearing capacity and settlement of footings on rock are influenced by factors such as the presence, orientation, and condition of discontinuities, weathering profiles, and other geological features. Therefore, the methods used for designing footings on rock should consider these site–specific factors.

For footings on competent rock, simple and direct analyses based on uniaxial compressive rock strengths and Rock Quality Designation (RQD) may be applicable. Competent rock is defined as a rock mass with tight discontinuities or those that are not wider than 3.5 mm. For footings on less competent rock, more detailed investigations and analyses should be conducted to account for the geological complexity of the rock. Below are comments on the methods used for footings on both competent and jointed rock.

Footings on competent rock

The allowable contact stress for footings supported on level surfaces in competent rock may be determined using the method proposed by Peck *et al.* (1974). However, the maximum allowable contact stress must not exceed the concrete's allowable bearing stress. RQD used in this method should be the average RQD for the rock within a depth of B below the base of the footing, assuming the RQD values are relatively uniform within that interval. If the rock mass within a depth of $0.5B$ below the base of the footing is of poorer quality, the RQD of the poorer rock should be used to determine the allowable contact stress (q_{all}).

Footings on jointed rock

Using the uniaxial compressive strength: the design of footings on broken or jointed rock must account for the condition and spacing of joints and other discontinuities. The ultimate bearing capacity of footings on broken or jointed rock may be estimated using the following equation (1) (Hoek, 1983):

$$q_{ult} \cong N_{ms} q_{ucs} \qquad (1)$$

The values of q_{ucs} should preferably be determined from the results of laboratory testing of rock cores obtained within $2B$ of the base of the footing. The coefficient N_{ms} is a function of the rock category and rock type. When the rock strata within this interval vary in strength, the rock with the lowest capacity should be used to determine q_{ult}. In the design example case presented in this paper, there are uniaxial compressive test results available from gabbro, which give a conservative mean of 170 MPa. This value is higher than the minimum value of 124-311 MPa (AASHTO, 2002) but lower than the mean of the two values. Nonetheless, it is representative of the rock mass and can be used to estimate the probable ultimate bearing capacity of the rock mass in situ.

Using the rock mass rating (RMR), another empirical–theoretical method to estimate the net allowable bearing capacity of a fractured rock mass is based on the use of the Rock Mass Rating of Bieniawski (1989) RMR_{89} system. In this study, the RMR_{89} of the gabbro within the probable pressure bulb has a mean value of 90%, which corresponds to a net allowable bearing capacity in the range of 4 to 6 MPa.

Using RQD, Peck *et al.* (1974) assessed the allowable bearing capacity (q_{all} in MPa) directly from the RQD using equation (2), obtained in borings or field measurements (Palmström's method). This assessment assumes that the applied stress will not exceed the uniaxial compressive strength (UCS) of the intact rock ($q_{all} < q_{UCS}$). This approach, as noted by many investigators, often results in values that are higher compared to other methods.

$$q_{all} = 1 + \frac{RQD/16}{1 - \left(RQD/130\right)} \tag{2}$$

Drawing from Canadian experience, another method to estimate the allowable bearing capacity of rock under pressure is detailed in the Canadian Foundation Manual (CFM, 2006). Developed by Gill (1980), this method incorporates the uniaxial compressive strength of the intact rock along with factors that account for various rock mass characteristics, foundation types, and their representative dimensions. The Canadian practice method is applicable for socketed piles and shallow foundations (Gill, 1980), employing equation (3) for calculating the allowable bearing capacity of the rock mass:

$$q_{all} = q_{UCS} N_j N_d \tag{3}$$

where N_j is an empirical coefficient depending on the spacing of the discontinuities and N_d is an empirical coefficient depending on the embedment of the foundation using equations (4) and (5).

$$N_j = \frac{3 + s/B}{10\sqrt{1 + (300\delta/s)}} \tag{4}$$

$$N_d = 0.8 + 0.2\left(\frac{h}{D}\right) < 2 \tag{5}$$

Where s is the spacing of joints in cm, B is the footing width in cm, δ is the opening of joints in cm, h is the embedment and D is the embedment in rock. The method states that normally $N_d \geq 1.0$ and that for shallow foundations the engineer must consider that $N_d = 1$.

In the present design example and using the last method it was determined the following rock mass characteristics: s between 20 – 45 cm (average value = 30 cm), B between 2.00 – 4.00 m (average value = 300 cm), and δ between 0.10 – 0.4 cm (average value = 0.3 cm). With these parameters, N_j is 0.155, and N_d is 1.

A method developed by Serrano and Olalla (1994) gives an ultimate load capacity P_{ult} that can be estimated with equation (6):

$$P_{ult} = \beta(N_\beta - \zeta) \tag{6}$$

This approach considers two variables β in MPa and ζ, equations (7) and (8). Where m, s and m_i are the Hoek and Brown failure criterion parameters and σ_{ci} is the uniaxial compressive strength of the intact rock. The bearing capacity factor N_β is a generalization of the Prandtl parameters N_c and N_q, and it is a function of the ground slope, of the angle of the load and the normalized external overburden acting around the footing.

$$\beta = \frac{m\sigma_{ci}}{8} = \frac{\sigma_{ci}}{8} m_0 \exp\left(\frac{RMR - 100}{28}\right) \tag{7}$$

$$\zeta = \frac{8s}{m^2} = \frac{8}{m_i^2} \exp\left(\frac{RMR - 100}{25.2}\right) \tag{8}$$

Following the procedure by Wyllie (2003), a practical approach can be used to estimate the foundation's bearing capacity. The mechanism assumes that an active wedge forms below the footing and interacts on a passive wedge extending to the side (Salgado, 2022). The rock under the foundation (zone A) and the contiguous rock (Zone B) are assumed to be in compression similar to a specimen in a triaxial compression (see Figure 3), with major principal stress (σ_{1A}, σ_{1B}) and minor principal stress (σ_{3A}, σ_{3B}). For a footing resting above the rock σ_{3B} = 0. For a recessed footing the surcharge q_s is the average vertical stress due to the rock weight above the footing level. The increase in the bearing capacity for this case is produced by the confining pressure. The fracture rock strength is defined by the Hoek–Brown criterion with the constants m and s to account for the rock mass fracturing. The intact rock strength $\sigma_{u(r)}$, is determined from laboratory tests (unconfined compressive strength) on rock cores. The major principal stress in the zone A can be related to the ultimate bearing stress, equations (9) and (10).

$$\sigma_{1A} = q_u = \sqrt{m\,\sigma_{u(r)}\sigma'_3 + s\sigma^2_{u(r)}} + \sigma'_3 \tag{9}$$

$$\sigma'_3 = \sqrt{m\,\sigma_{u(r)}q_s + s\sigma^2_{u(r)}} + q_s \tag{10}$$

The allowable bearing capacity q_{all}, see equation (11), relates to the rock mass strength by the factor of safety FS (between 2 and 3) and the correction factor of foundation shapes C_{f1} = 1, *e.g.* for a stripe L/B > 6 (Wyllie, 2003).

$$q_{all} = C_{f1}q_u/FS \tag{11}$$

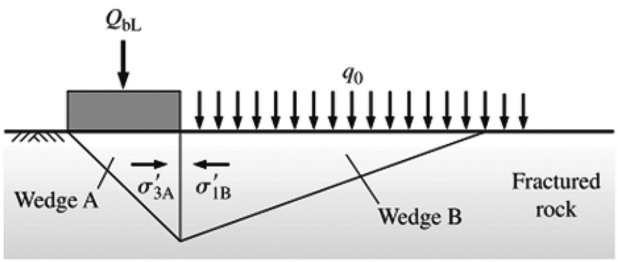


Figure 3: Bearing capacity failure mode (Salgado, 2022)

Dynamic considerations

The dynamic shear strength parameters can be estimated based on their static counterparts because conducting dynamic tests on a rock mass with an RQD greater than 30% is impractical, and even less feasible for an RQD below 25%, which Bieniawski (1989) classifies as resembling dense coarse granular soil.

In this study, it is assumed that the rock mass may behave similarly to granular soils under dynamic loads. Referring to research on Ottawa sand by Whitman and Healy (1963), it is noted that increasing load velocity (deformation velocity) leads to a slight reduction in the internal friction angle. For practical purposes, these researchers propose the following relationship to estimate the dynamic friction angle of granular soils using equation (12):

$$\phi_{dyn} = \phi_{static} - 2° \tag{12}$$

To estimate the cohesion under dynamic conditions, the authors referenced investigations on stress–strain behaviour characteristics of granular and fine soils under transient loading (Casagrande and Shannon, 1949; Carroll 1963). Following Carroll's proposal we can use equation (13):

$$c_{u\,transient} \cong 1.5c_{u\,static} \tag{13}$$

The correlation between dynamic and static deformability parameters is theoretically straightforward. However, challenges arise when engineers encounter heterogeneous materials like certain soils and rock masses, especially when fractures have separations comparable in size to the foundations' dimensions. In such cases, treating the material as a perfect continuum can lead to theoretical parameter estimates that rarely match those from empirical methods.

Experience has led engineers to prefer adopting static values estimated by empirical methods based on the research of past scholars (Hoek and Brown, Bieniawski). These methods allow for the inclusion of deformability and shear strength parameters in the estimation process for dynamic values in the static case. Engineers then adjust these values based on empirical coefficients derived from laboratory results that consider the material type (granular) and scaling factors.

These considerations provide a practical degree of validity. Therefore, values determined using mechanics equations (dynamics) should only serve as reference values for an idealized rock mass and may not fully reflect the reality of the site project.

Design example on igneous rock

The study will analyze the behaviour of fractured rocks using samples from project sites located in northern Chile. The site primarily features gabbro, an igneous rock known for its mafic composition, dark color, and phaneritic, intermediate to coarse–grained texture. The example structure presents a unique foundation design scenario, involving an excavation with a central recessed footing and two recessed ring footings shown in Figure 4. The foundation's concrete is considered to have a minimum strength of 40 MPa. This case aims to compare results obtained from empirical methods with those derived from a numerical model using the finite element method in RS2 (v11.023) software by Rocscience, Inc.

Specifically, the central footing or pier measures 4.5 m in base width at a depth of 7.0 m. The first ring footing is situated 12.0 m away, with dimensions of 4.0 m width and 4.5 m depth. The second ring footing is located 30.0 m away, featuring a width of 2.0 m and a depth of 2.5 m.

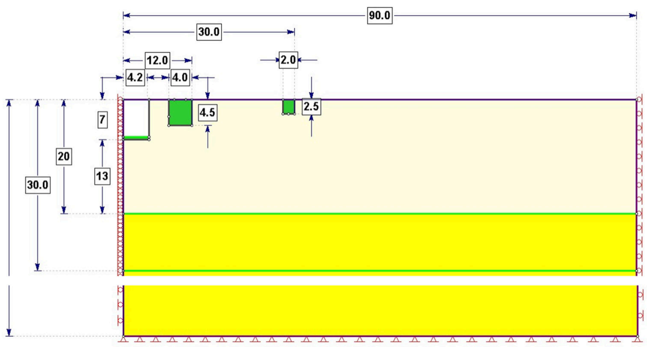


Figure 4: Design example. Foundation with a pier and two foundation rings on recessed on a fractured igneous rock.

Normally, rocks have very high bearing capacities, and in the case of igneous rocks, they can exceed the compressive strength of concrete. In such cases, the allowable bearing capacity is determined not by the properties of the rock mass but by the strength of the concrete. Moreover, settlements can be so minimal that they qualify as elastic deformations and are often negligible in many instances.

Empirical methods

Based on the methods discussed earlier, Tables 1 and 2 present a comparison of estimated values for the ultimate and allowable bearing capacity of the fractured rock mass in the design example.

FEM model

The numerical model was implemented using the software RS2 by Rocscience Inc. The analysis type applied to this model was axisymmetric, and the solver type was Gaussian Elimination. All materials are considered isotropic, and the failure criterion is Mohr–Coulomb. The mesh type is graded, and the element type are 6 noded triangles. The number of elements is 1992 and the number of nodes is 4113. The design loads for each foundation are 4 MPa. The seismic coefficient k_h = 0.4g in the Chilean seismic zone 3. The material properties are shown in Table 3 and the corresponding model is shown in Figure 5. There are 3 zones of rock quality, up to 20 m gabbro 1 (highly fracture rock), 20 to 30 m gabbro 2 (slightly fracture rock) and 30 m and beyond is gabbro 3 (good to very good rock).

Table 1: Comparison of the estimated values of the ultimate and allowable bearing capacity of the fractured rock mass of the design example (Part 1).

Method	UCS AASHTO	RMR_{89}	RQD	Serrano and Olalla
$q_{u,\ static}$, MPa	39.1	6.2	17.7	11.5
$q_{u,\ dynamic}$, MPa	58.7	9.3	26.6	17.3
$q_{all,\ static}$, MPa	13.0	2.1	5.9	3.8
$q_{all,\ dynamic}$, MPa	19.5	3.1	8.9	5.8

Table 2: Comparison of the estimated values of the ultimate and allowable bearing capacity of the fractured rock mass of the design example (Part 2)

Method	CFM Central Pier	CFM Ring 1	CFM Ring 2	Wyllie Central Pier	Wyllie Ring 1	Wyllie Ring 2
$q_{u,\ static}$, MPa	77.4	78.4	80.3	40.2	37.4	33.9
$q_{u,\ dynamic}$, MPa	116.0	117.6	120.5	60.4	56.1	50.9
$q_{all,\ static}$, MPa	25.8	26.1	26.8	16.8	12.5	11.3
$q_{all,\ dynamic}$, MPa	38.7	39.2	40.2	25.2	18.7	17.0

Results

Upon examining the results obtained from pseudo–three–dimensional analyses through numerical methods, it can be stated that the contact pressures or stresses, even under significant seismic demand, are comparable to the values proposed for the admissible load capacity of the rock mass by the RMR_{89} method (lower bound). This method is the most conservative among all empirical methods. The results, derived from the parameters of the rock mass estimated through surface mapping, point load tests, and calibrated through simple compression tests using the latest version of Rocscience's RSData program (Hoek–Brown failure criteria), support the recommendation to use these values. Although conservative, these values allow for an acceptable cross–check with numerical methods (see Tables 1 and 2 and Figures 6 and 7).

Table 3: Material properties used in the design example for the foundation and rock mass

Material	Foundation concrete	Gabbro 1	Gabbro 2	Gabbro 3
Initial element loading	Body force only	Field stress and body force	Field stress and body force	Field stress and body force
Unit weight, MN/m^3	0.030	0.025	0.025	0.025
Poisson's ratio	0.3	0.12	0.16	0.18
Elasticity modulus, MPa	28500	8241	47486	103299
Material type	Plastic	Elastic	Elastic	Elastic
Peak tensile strength, MPa	0.8	0	0.4	2.7
Peak friction angle, degrees	44	24	39	48
Peak cohesion, MPa	8	10	20	32

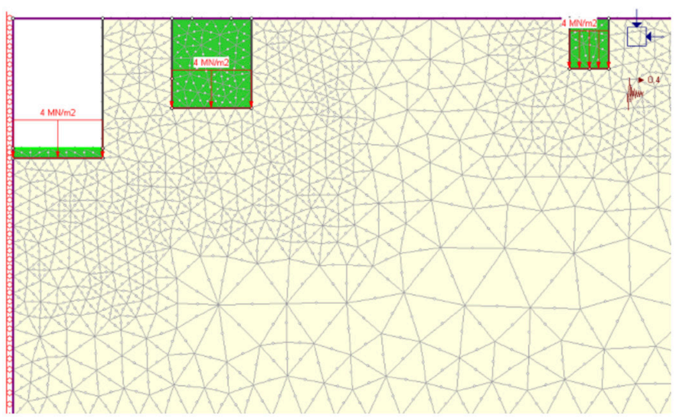


Figure 5: Design example. Mesh, loads and seismic coefficient.

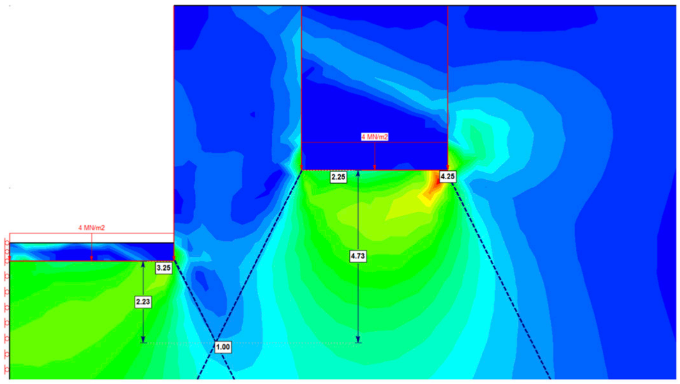


Figure 6: Design example. Differential stresses (MPa) induced by static loads and the determination of interactions among the ring foundation elements.

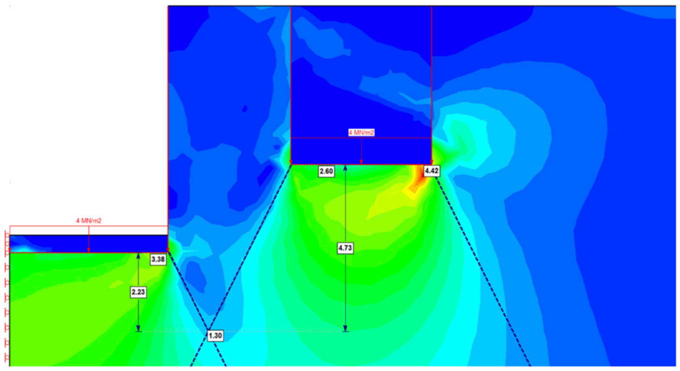


Figure 7: Design example. Differential stresses (MPa) induced by seismic loads and the determination of interactions among the ring foundation elements.

Additionally, considering an approximate dissipation of deviatoric stresses (expressed clearly as maximum shear stresses) below the foundations, it is shown that these stresses do not reach levels that would be considered high risk for the rock mass beneath the foundation.

Moreover, the vertical deformations of the system under static load conditions are within the established limits for maximum deformations, set at 5 mm. This also leads to a recommendation to use the RMR_{89} method in similar cases (in other rock masses) to estimate the admissible bearing capacity of a rock mass (see Figures 8 and 9).

Finally, even under significant seismic demand, both the stresses and deformations remain within the established limits for the foundation system of the structures. In the case of deformations, there is even a slight increment (compare Figures 8 and 9). Therefore, among the empirical methods, the RMR_{89} method is the most aligned with reality. As

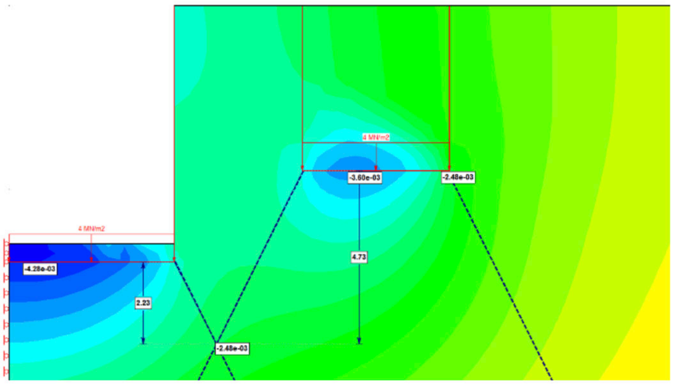


Figure 8: Design example. Vertical displacements (mm) induced by static loads and the determination of interactions among the ring foundation elements

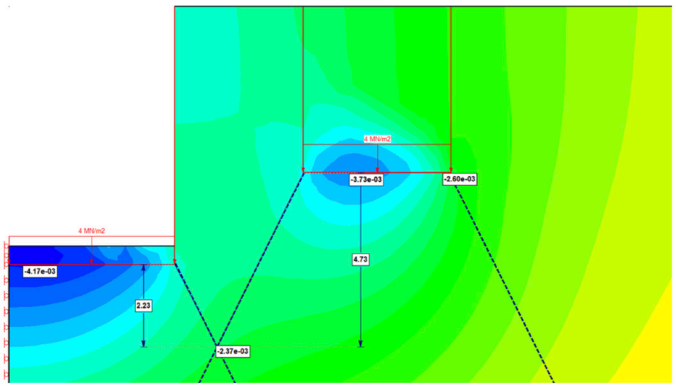


Figure 9: Design example. Vertical displacements (mm) induced by static loads and the determination of interactions among the ring foundation elements

shown by the results in Table 1, it is the most conservative method available in the literature.

The maximum allowable contact stress must not exceed the concrete's allowable bearing stress (upper bound limit). Normally, rocks have very high bearing capacities, they can exceed the compressive strength of concrete. In this case, the allowable bearing capacity is determined not by the properties of the rock mass but by the strength of the concrete.

Conclusions

Under static load conditions, the stresses calculated do not reach levels that would be considered high risk for the rock mass beneath the foundation. Vertical displacements of the foundation system do not exceed 5 mm.

Under high seismic demand, both the stresses and displacements still remain within the established limits for the foundation system, although there is a slight displacement increment. Therefore, it can be concluded that among the empirical methods, the RMR_{89} is the most conservative method. In this analysis, the allowable bearing capacity is controlled by the strength of the concrete and not by the properties of the rock mass.

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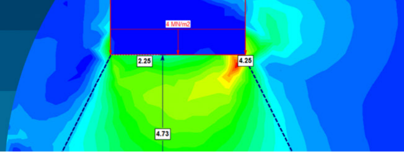

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