Proceedings of the 17th Pan-American Conference on Soil Mechanics and Geotechnical Engineering (XVII PCSMGE), and 2nd Latin-American Regional Conference of the International Association for Engineering Geology and the Environment (IAEG), La Serena Chile, 2024.

The design of shallow foundations on fractured rock

Diseño de fundaciones superficiales en roca fracturada

Néstor René Espinoza Guillén
Escuela de Ingeniería Civil, Universidad de Valparaíso, Chile, rene.espinoza@uv.cl

Jorge A. Arriagada Triana
Escuela de Ingeniería Civil, Universidad de Valparaíso, Chile, jorge.arriagada@uv.cl

Lorna González Martínez
NREG Consultores en Geotecnia, Chile, lorna.gonzalezm@alumnos.uv.cl

Karime Nazer Flores
Laboratory Manager, IDEAS, Institute of Testing, Analysis and Drilling, Chile, karime.nazer@alumnos.uv.cl

ABSTRACT: 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 modeling, no universal solutions exist. This study analyzes fractured rock behavior 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

1 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.

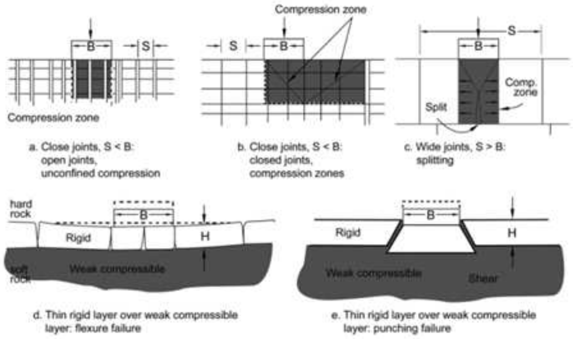

Figure 1. Bearing capacity failures modes (Sowers and Sowers 1979).

A simplified representation of foundations transitioning from intact to heavily jointed rock mass with increasing sample size is presented by Serrano and Olalla (1996) (as shown in Figure 2), which is a modified scheme based on the idealized diagram by

Hoek (2023). This representation illustrates the influence of scale on the rock mass behavior 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.

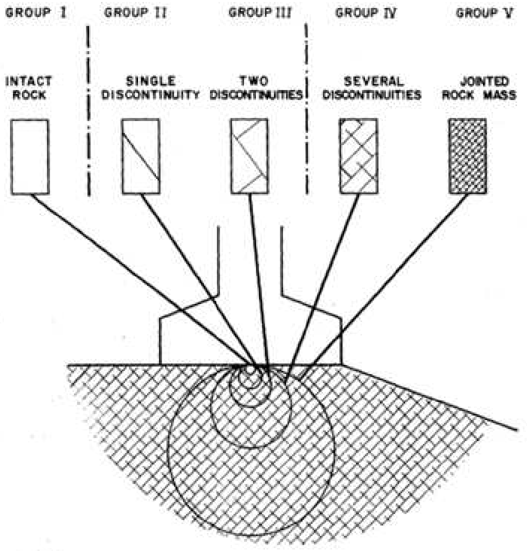

Figure 2. Simplified representation of the influence of scale on type of rock mass behavior model which should be used in designing shallow foundations on rock slope. Modified from (Serrano and Olalla 1996)

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.

2 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. 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:

2.1 *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. The Rock Quality Designation (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}).

2.2 *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 relationship (Hoek 1983):

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

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 tests 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. The correlation with the net allowable bearing capacity was by Singh (1991) and Mehrotra (1992).

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 the Rock Quality Designation (RQD), Peck et al. (1974) assessed the allowable bearing capacity (q_{all} in MPa) directly from the RQD 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.

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

Drawing from Canadian experience (CFM), another method to estimate the allowable bearing capacity of rock under pressure is detailed in the Canadian Foundation Manual. 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 a straightforward formula for calculating the allowable bearing capacity of the rock mass:

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

Where Nj is an empirical coefficient depending on the spacing of the discontinuities and Nd is an empirical coefficient depending on the embedment of the foundation.

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

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

Where s is the spacing of joints in centimeters, B is the footing width in centimeters, δ is the opening of joints in centimeters, 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, Nj 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 by the following expression:

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

This approach considers two variables β in MPa and ζ. Where m, s and mi 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.

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

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

Following the procedure by Wyllie (1999), 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 $\sigma_{3B} = 0$. 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.

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

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

The allowable bearing capacity q_{all} 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 1999).

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

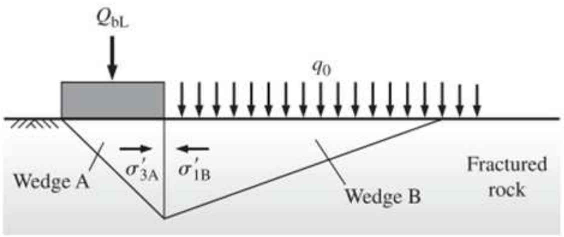

Figure 3. Bearing capacity failures modes (Salgado 2022)

2.3 *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 Lambe 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:

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

To estimate the cohesion under dynamic conditions, the authors referenced investigations on stress–strain behavior characteristics of granular and fine soils under transient loading (Casagrande and Shannon 1949, Carroll 1963). Following Carroll's proposal, the practical relationship is as follows:

$$c_{u\,transient} \cong 1.5 \cdot c_{u\,static} \qquad (13)$$

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.

3 DESIGN EXAMPLE ON IGNEOUS ROCK

The study will analyze the behavior 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. 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.

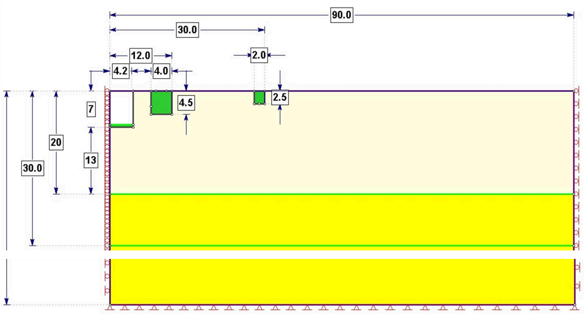

Figure 4. Design example. Foundation with a pier and two foundation rings on recessed on a fractured igneous rock.

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.

3.1 *Empirical methods*

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

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

3.2 *FEM Model*

The numerical model was implemented using 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).

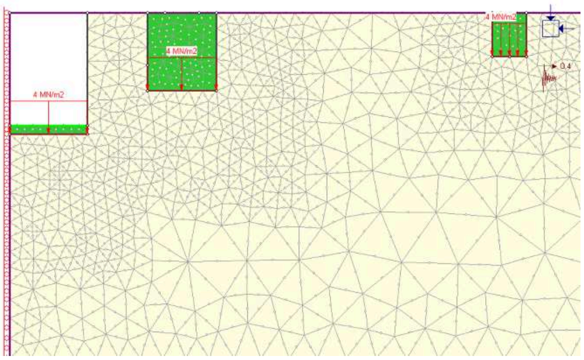

Figure 5. Design example. Mesh, loads and seismic coefficient.

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/m3)	0.030	0.025	0.025	0.025
Poisson’s ratio	0.3	0.12	0.16	0.18
Elasticity modulus (MPa)	28,500	8,241	47,486	103,299
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

4 RESULTS AND CONCLUSIONS

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 Table 1, Table 2 and Figures 6 and Figure 7)

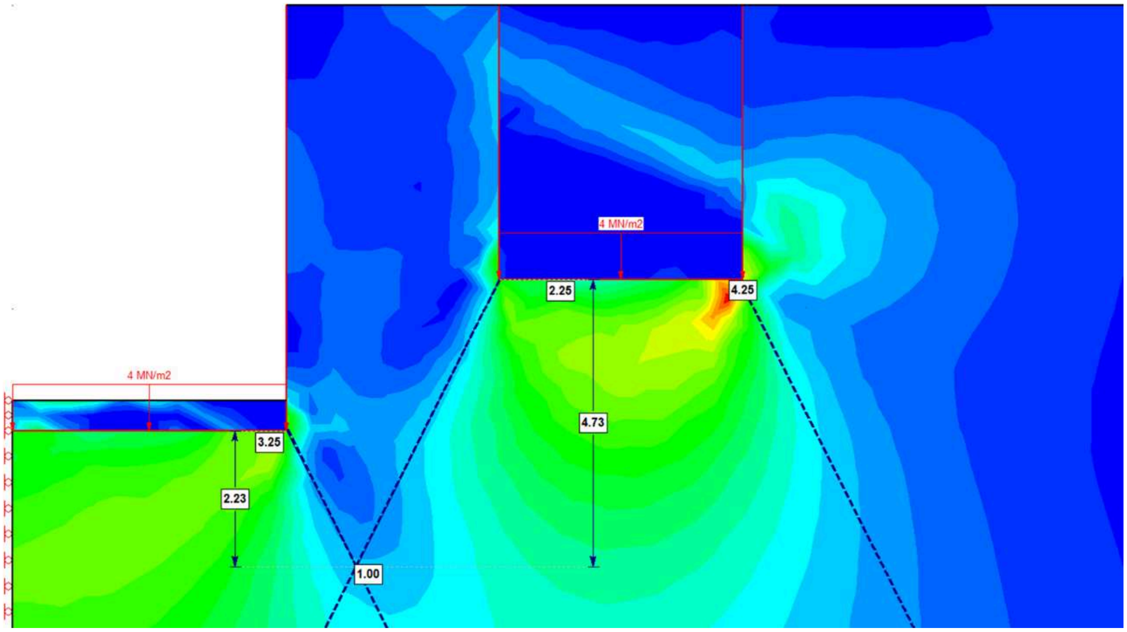

Figure 6. Design example. Differential stresses (MPa) induced by static loads and the determination of interactions among the ring foundation elements.

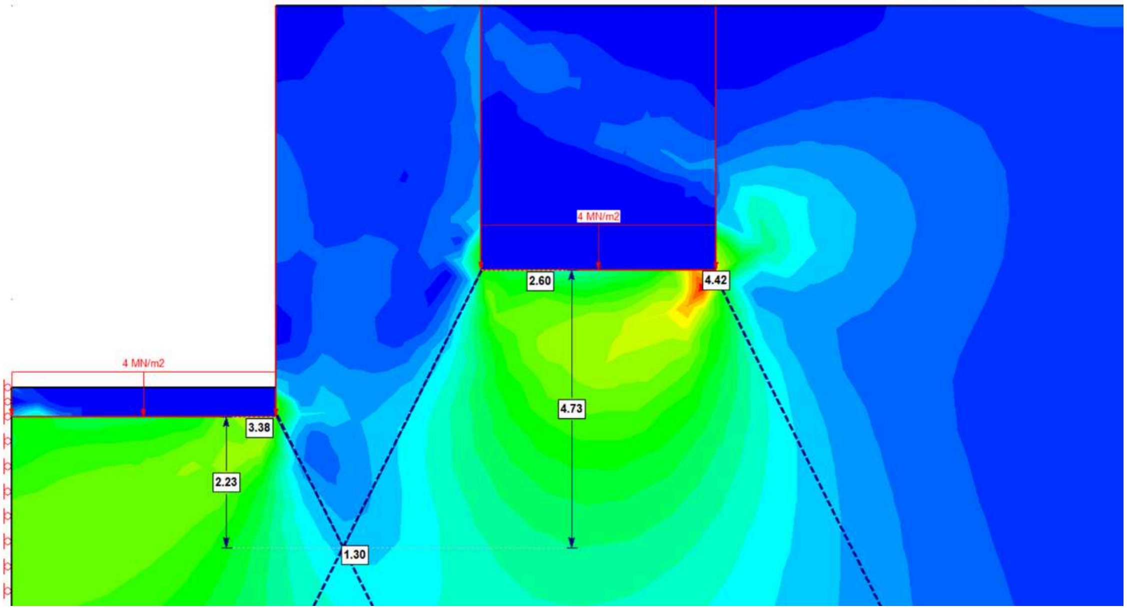

Figure 7. Design example. Differential stresses (MPa) induced by seismic loads and the determination of interactions among the ring foundation elements.

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 Figure 8 and Figure 9).

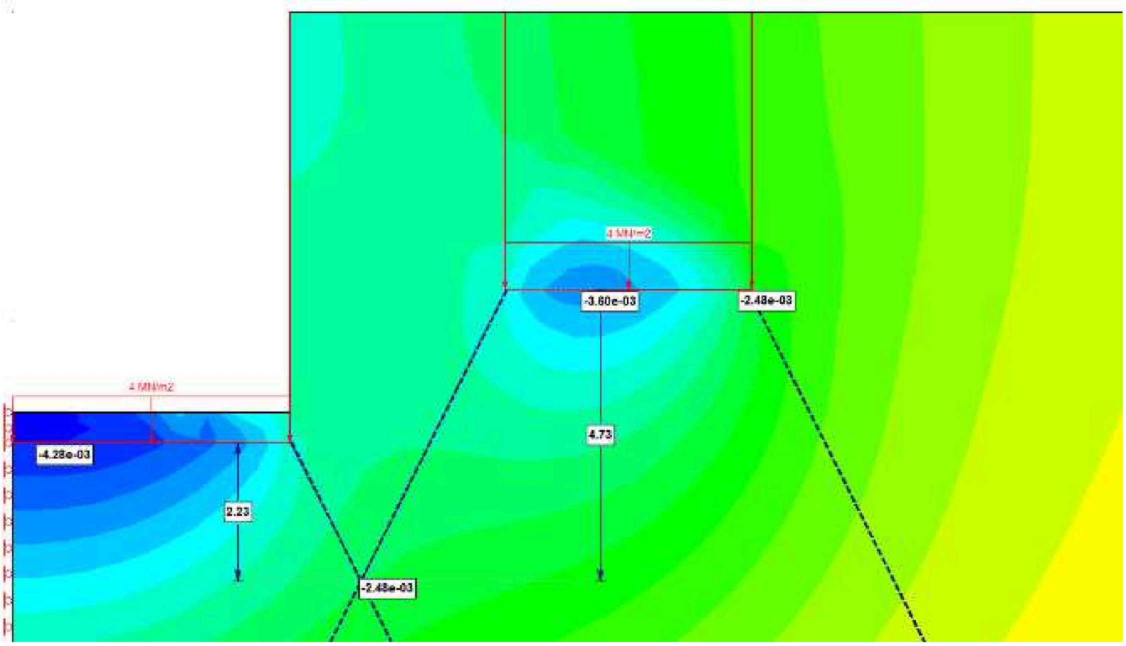

Figure 8. Design example. Vertical displacements (mm) induced by static loads and the determination of interactions among the ring foundation elements

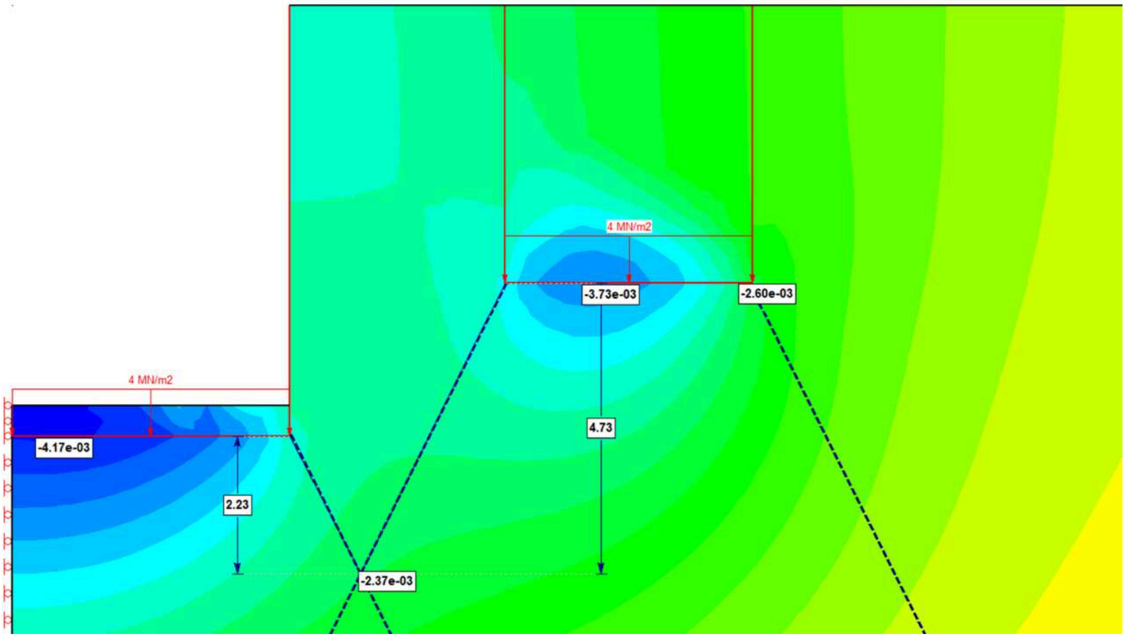

Figure 9. Design example. Vertical displacements (mm) induced by static loads and the determination of interactions among the ring foundation elements

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 Figure 8 and Figure 9). Therefore, it can be concluded that among the empirical methods, the RMR_{89} method is the most aligned with reality. As 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.

5 REFERENCES

AASHTO, ed. 2002. Standard Specifications for Highway Bridges. 17. ed. Washington, DC: American Association of State Highway and Transportation Officials.
Aydan, Ömer. 2017. Rock Dynamics. ISRM Book Series, volume 3. Leiden, The Netherlands: CRC Press/Balkema.
Feng, Xiating, and J. A. Hudson. 2011. Rock Engineering Design. Leiden, The Netherlands: CRC Press/Balkema.
Goodman, Richard E. 1989. Introduction to Rock Mechanics. 2nd ed. New York: Wiley.
Hoek, E. 2023. Practical Rock Engineering.
Rose, Andrew. 2004. "RMR Rock Properties for Shallow Foundation Design." Journal of Engineering Technology 21 (September): 42–50.
Salgado, Rodrigo. 2022. The Engineering of Foundations, Slopes and Retaining Structures. Second edition. Boca Raton, FL: CRC Press, Taylor & Francis Group.
Serrano, A, and C. Olalla. 1994. "Ultimate Bearing Capacity of Rock Masses." International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts 31 (2): 93–106.
———. 1996. "Allowable Bearing Capacity of Rock Foundations Using a Non-Linear Failure Criterium." International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts 33 (4): 327–45.
Sowers, George F., and George B. Sowers. 1979. Introductory Soil Mechanics and Foundations: Geotechnical Engineering. 4th ed. New York: Macmillan.
Wittke, Walter. 2014. Rock Mechanics Based on an Anisotropic Jointed Rock Model, AJRM. Berlin: Wilhelm Ernst & Sohn.
Wyllie, Duncan C. 2003. Foundations on Rock. 0 ed. CRC Press.