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Validation and Numerical Simulation of the Bearing Capacity of Strip Footing on Sedimentary Jointed Rocks

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ملخص
بناء على نتائج حقلية مستخلصة من اختبار تحميل لوحي تم تنفيذه في موقع سد بختاري بإيران عام 2012، تمت دراسة أقصى حمل تتحمله الصخور المتكونة من حجر جيري رسوبي به فواصل تحت أساسات السد. وباستخدام طريقة العناصر المحددة تم بناء نموذج عددي بهدف محاكاة تأثير أقصى حمل للاختبارعلى الصخور. وقد تم بناء النموذج مع الاخذ بالاعتبار الشروط الحدودية للنموذج وجساءة قرص التحميل وتمثيل كتلة الصخر مفصلة كصخر سليم وفواصل تفصل بين الكتل السليمة. وتم استخدام نتائج الاختبار بهدف معايرة وتأكيد النموذج العددي. وقد وجد أن هناك نسبة بين خصائص الصخر السليم والفواصل الفاصلة بين كتله، وقد تم استنتاج هذه النسبة بين معامل المرونة لصخر السليم والمواد المالئة للفواصل، وكذلك استنتاج النسبة بين قوة التماسك بينهم. ومن خلال استخدام تلك النسب في تمثيل الاختبار عدديا، وجد أنها على توافق مع النتائج الحقلية بنسبة كبيرة ويمكن الاعتماد عليها في تمثيل كتلة الصخر مفصلة في النموذج كما هو متواجد في حالته الطبيعية. بالإضافة الى النسبة المستنتجة، تم عمل دراسة بارامترية بتغيير تلك النسبة الى عدة نسب من 0.01 إلى 0.001 ودراسة تأثيرها على الحمل المسموح الذي تتحمله كتلة الصخر وذلك بوجود فواصل تميل بزاوية 30 و45 درجات على المستوى الأفقي.

Abstract
Available results from plate load test performed in the Bakhtiary Dam in Iran were utilized to investigate the Bearing Capacity of strip footings on Sedimentary Jointed Rocks. A numerical model was developed using the finite element method to simulate and validate the Plate load test performed on Limestone formation in the dam site. The numerical model was formed as a discrete model i.e. the intact rock and the joints, were modeled with their parameters individually, the predicted ratio between the shear strength parameters, the modulus of elasticity of the intact rock and the discontinuities showed a reasonable agreement with the measured settlements values and the conducted results from the numerical model. The results emphasize the significant effect of the cohesion ratio between joints and the intact rock, which is the main factor affecting the bearing capacity of the rocks. A parametric study had been carried out with different ratios, to show the effect of these ratios with presence of inclined joint (30,45, and nearly vertical 70 degrees) on the allowable bearing capacity of the rock mass.

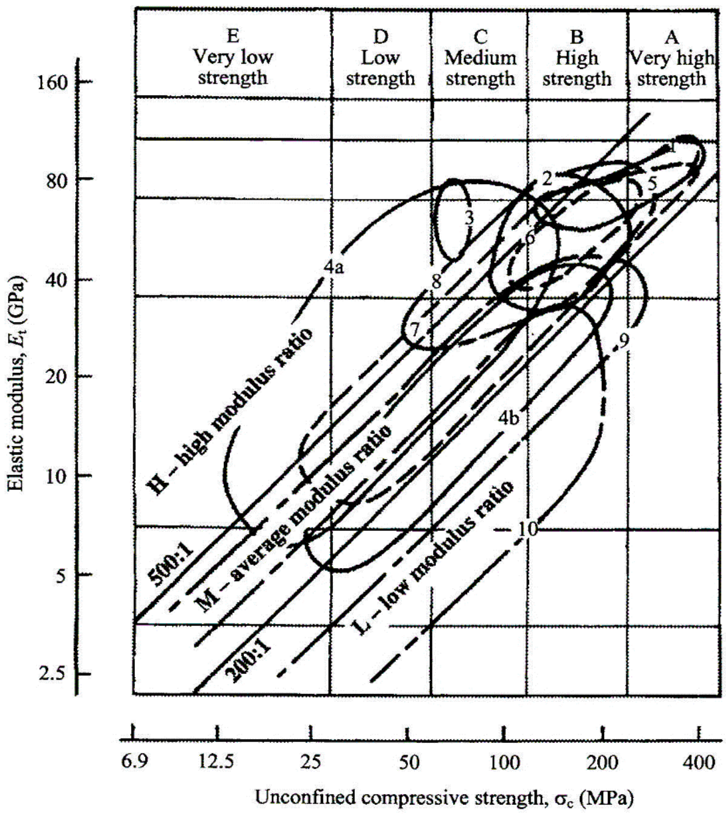
Keywords
Sedimentary Rocks; Jointed Rock; Bearing Capacity; Rock Mass Rating; Finite Elements analysis; Mohr-Coulomb Criteria; Joint dip angles.

1- INTRODUCTION
As result of the increasing urban development in Egypt in the last few decades, the locations of some construction projects sometimes present in areas with special geological nature. Accordingly, the evaluation of the bearing capacity of jointed rocks has become one of the urgent topics in geotechnical engineering.

The rock mass is often considered of a heterogeneous nature that can be treated as a discontinuous medium composed of intact rock blocks separated by discontinuous planes, i.e. joints. As the well-known types of sedimentary rock, such as limestone, sandstone, shale or marl, have this heterogeneous criteria.

The bearing capacity of jointed rock mass is significantly dependent on the shear strength and stiffness of the rock material and the discontinuities, as well as the number, orientation and condition of the discontinuities. Through a numerical analysis and depending on conducting in-situ loading tests, a verification had been made of these results to conduct a ratio between the shear strength and the stiffness parameters of joints to the intact rock.

As the intact rock occupies the largest proportion of the rock mass, a linear behavior will govern the mass. The linear deformation of the mass is in direct link with the significant cohesion of the rock, (Deere and Miller, 1966) published a classification system showed in
Figure (**23**), depending on the unconfined compressive test and the modulus of elasticity Es at 50% of the ultimate strength of the intact rock.



Metamorphic:
1 Quartzite
2 Gneiss
3 Marble
4a Schist, steep foliation
4b Schist, flat foliation

Igneous:
5 Diabase
6 Granite
7 Basalt and other flow rocks

Sedimentary:
8 Limestone and dolomite
9 Sandstone
10 Shale

Figure (23): Engineering classification of rock by Deformation modulus, (adapted from Deere and Miller, 1966)

As result, a linear perfect plastic model was established using Mohr-Coulomb criteria to simulate the rock mass and conduct a ratio between the joints and the intact rock stiffness and shear strength parameters.

2- CASE STUDY

According to (Agharazi et al, 2012) Bakhtiary dam and hydroelectric power plant project includes the design and construction of a 315m high, double curvature concrete dam and an underground powerhouse, with a nominal capacity of 1500MW, in the Zagros mountains in southwest Iran. The dam abutments are laying on medium to thickly bedded lightly deformed dark gray limestone.

Agharazi (Agharazi et al, 2012) carried intensive laboratory tests to determine the different parameters for both intact and joints material. Accordingly, uniaxial compressive tests on cores extracted from the site showed an average of 125 MPa in dry condition and 110 MPa for saturated samples. Yung’s modulus of intact rock was determined from the linear part of the axial stress-strain curve Ei =70 GPa for all Plate load test.

Twenty six (26) plate load tests were carried out in the dam’s site, categorized into groups with respect to the loading direction to the joints orientation i.e. NB normal to bedding, NJ1 normal to joint set 1. One test was chosen to be simulated in the numerical model by 2D finite element code, and to be conducted the ratio between the stiffness, strength parameters of the intact and the separated joints. Figure (24) shows the test configuration and the related stress settlement curve, it is worth to mention that loading process carried as cyclic loading, and the maximum stress subjected to the rock mass was 20 MPa.

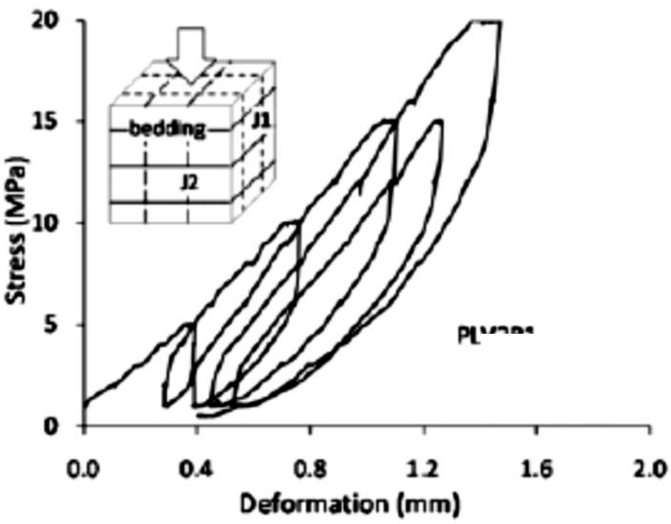


Figure (24): Configuration of the field PLT test (NB) normal to the bedding (after Agharazi et. al, 2012)

Table 7: The Resulted settlements/deformations under the loaded plate load test (after Agharazi et. al, 2012)

Loading Stress (MPa)	Deformations (mm)
5	0.40
10	0.52
15	0.76
20	1.09

The used plate load test was a rigid plate with a maximum diameter equal 971 mm, involving a hydraulic jack loading two opposite sides of test gallery to ensure reaching the maximum test load.

3- NUMERICAL MODELING OF THE CASE STUDY

In this paper, a numerical model is developed to simulate the latter mentioned case study (Agharazi et al, 2012). The main objective is to calibrate and validate the numerical model in order to provide a reliable numerical prediction of the rock mass behavior under the subjected stress. This paper partially focuses on the results of the numerical model calibration/validation. The numerical model is based on the finite elements method, and the simulation is carried out using the finite-element-based software PLAXIS 8.5 (Mohr-Coulomb model). The numerical model simulates the linear elastic-perfectly plastic behavior of rock mass during loading condition.

Figure 4) shows the finite element mesh used in the analysis. The numerical model established by setting boundaries equal to 6B (Weltman, 1983), B is the diameter of the plate taken as 1.00 m. The Joints were simulated as rigid interfaces inclined by 70 degrees on the horizontal plan (dip angle), this method is time-saving during the analysis comparing to be simulated as inclined joints as a cluster with a small width. The stress applied to the mass equal 20 MPa as distributed stress.

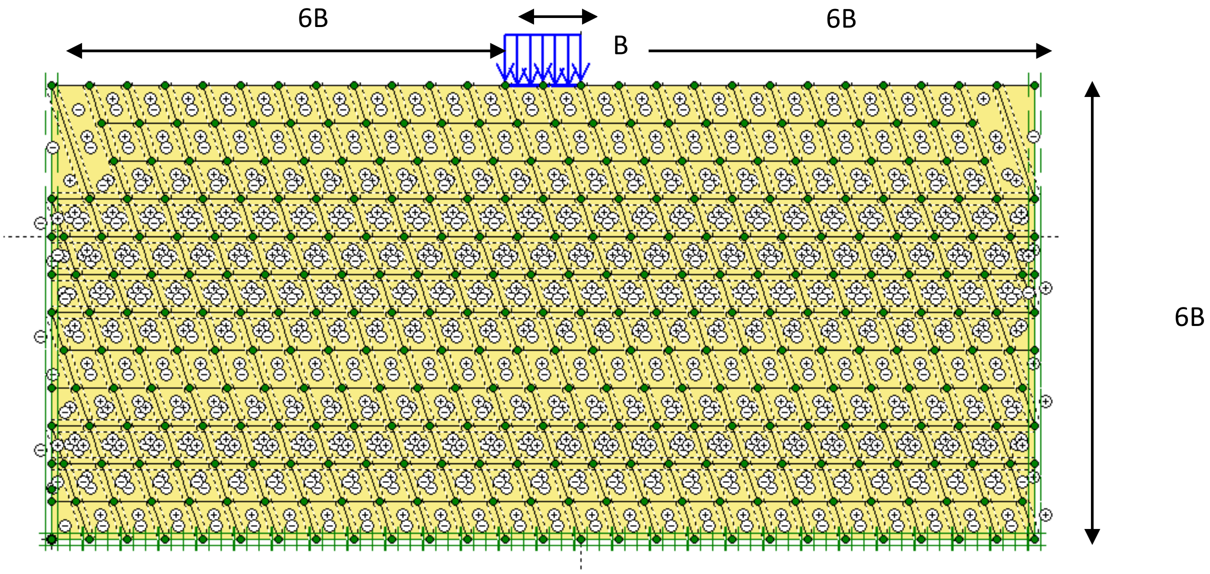


Figure (25): Configuration of the finite element model simulation.

It is worth to mention that the bedding (horizontal joints) are simulated also as a rigid interface with parameters approximately 90% of the intact rock stiffness and strength parameters, that can be explained as the bedding under the normal loading will close and increase the strength of the mass under the perpendicular loading i.e. no more critical shear zone failure as expected from the inclined joints.

The analysis carried out by input the intact rock parameters conducted from the case study and trying different ratios between the joint material and the intact rock material strength, stiffness parameters. The comparative result is the settlement from the Plate load test which, indicate 1.09 mm as a maximum settlement (see Figure (24).

Table 8: Geotechnical properties of the intact rock (after Agharazi et.al, 2012)

Property	Value
Unit weight (kN/m^3)	22
Internal friction angle (degrees)	35
Young Elastic modulus (GPa)	70
Cohesion (MPa)	62.5
RMR range	45-65

After reaching a reasonable ratio between the joint stiffness modulus and the intact modulus Ej/Ei, the previous steps were carried out again by changing the ratio between the cohesion ratio Cj/Ci, the internal friction angel kept 35° for the intact and 33° for the inclined joints, as no available data related to the roughness (asperity) of the joint's walls.

4- ANALYSIS RESULTS

As mentioned above, the stiffness and the shear strength of both the intact rock and the discontinuities are the dominating factors affecting the behavior of the entire rock mass. The numerical analysis results are illustrated with the field settlement as shown in Figure (26), by changing the ratio between the stiffness moduli, while keeping the cohesion ratio between the joints and the intact rock equal 0.05 as the C' for the joints equal 300 kPa.

Figure (26) shows that, as the stiffness ratio (Ej/Ei) gets smaller a large diverge observed between the field and the numerical results, this means that the joints are the main reason caused the failure. This sliding shear failure happens when the intact rock blocks slide along the weak joint's walls, which have a low cohesion as shown in Figure (27. Increasing the ratio Ej/Ei up to 0.15, the numerical results are in good agreement with the filed study as shown in Figure (26)

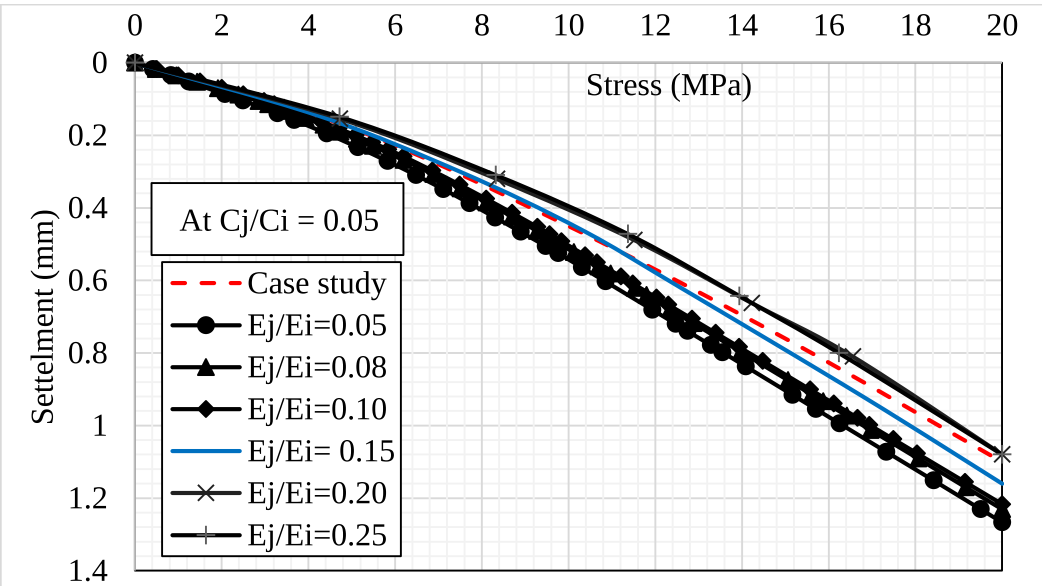


Figure (26): Field settlement and the numerical results with different E modulus ratios.

To evaluate the significant of fitting ratio 0.15, to be used further in models, the cohesion ratio was changed with keeping the stiffness modulus ratio equal 0.15 as performed and illustrated in Figure (28).

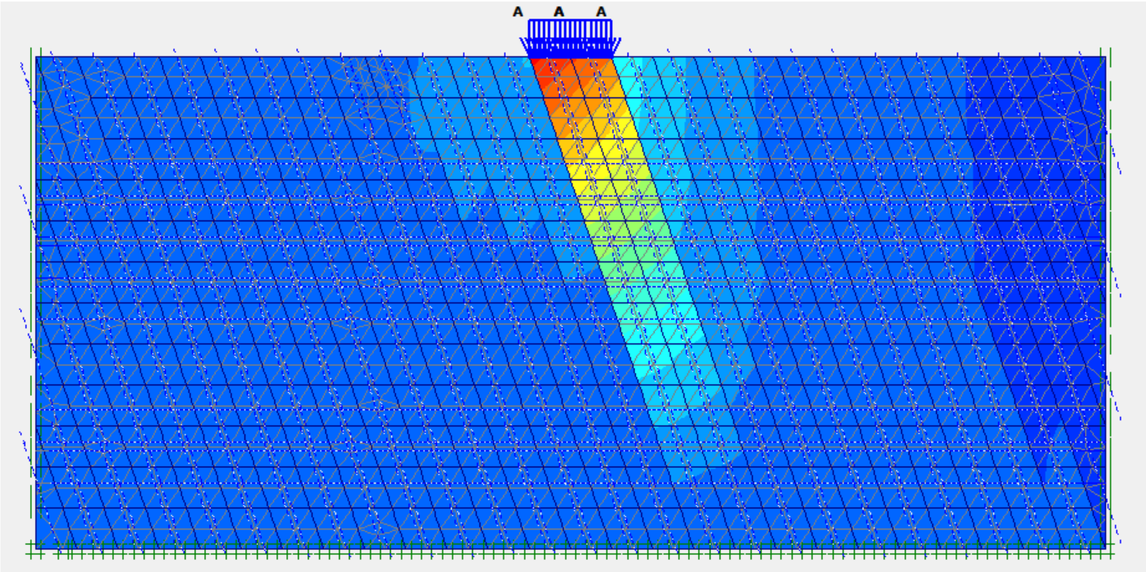


Figure (27) The failure mode; shearing of the intact blocks along the weak joint walls.

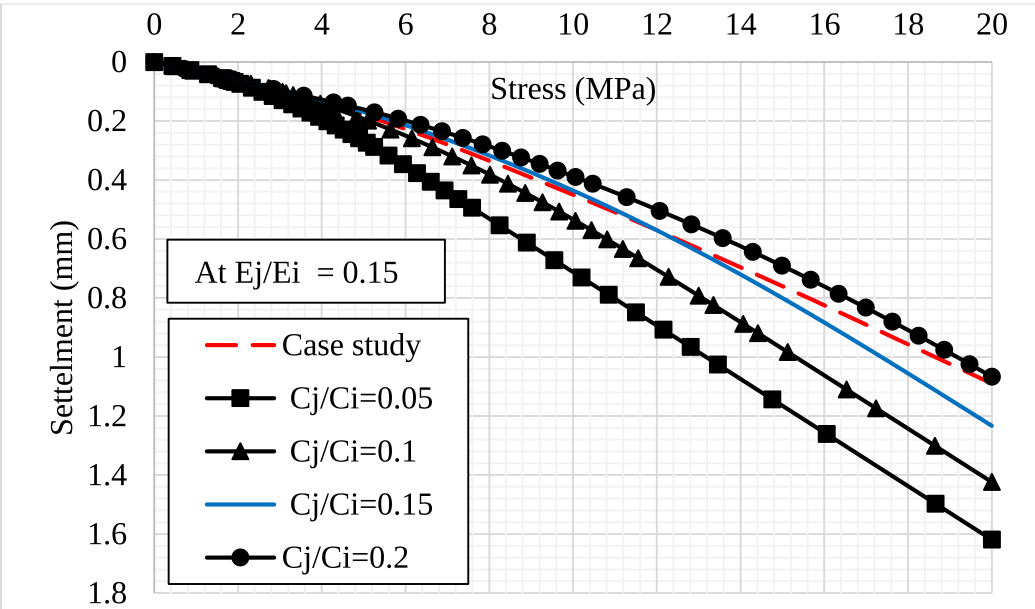


Figure (28): Field settlements results with the simulated numerical results with different cohesion and constant stiffness modulus

Figure (28) shows that most fitted ratio between the joint's cohesion "Cj" and intact rock cohesion "Ci" equal to 0.15, with Ej/Ei is kept constant and equal also 0.15.
As illustrated in the above figure, the joint's cohesion is an effective factor implemented the failure of the rock mass and deformation equally. This cohesion can be increased

according to the material filling this layer or the emptiness of the undulations of these joints.

5- PARAMETRIC STUDY

To investigate the effect of the conducted ratio of 0.15 between the strength parameters and the stiffness parameters of both the joints and the intact rock on the bearing capacity of the rock mass, a parametric study have been carried out especially when changing the dip angle of the joint to be less steeper than the validating model i.e. 30 and 45 degree with the horizontal plane.

To carry this study, two value of RMR were chosen as the upper value and the lower value recorded in the case study, 45° with joint spacing equal to 0.40, 2.00 m and RMR 65 with the same spacing. These two values can be classified as fair rock and good rock, respectively according to (Bieniawski, 1989).
Table 9 summarizes the parameters of the parametric study cases.

Table 9: Parameters of the Parametric study cases

RMR	E Gpa	C Mpa	Joint Spacing (m)
45 “Fair Rock”	11	2.75	0.4 and 2
65 “Good Rock”	20	5	

The studied dip angles are 30, 45, and 70 degrees. Every rock type had studied with two joint spacing 0.40 m and 2.00 m.
Figure (**29**) through Figure (32) show the effect of the joint dip angles on the allowable bearing capacity with respect of the ratios Ej/Ei and Cj/Ci.

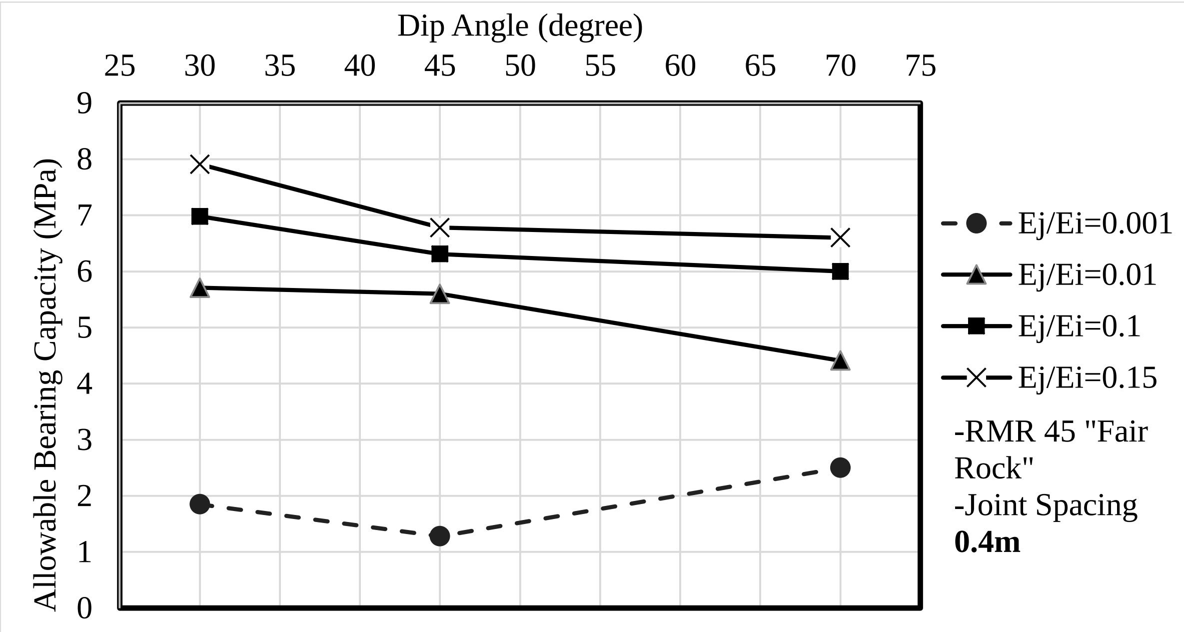


Figure (29): Effect of the joint dip angles on the allowable bearing capacity – RMR 45 – joint spacing 0.40 m

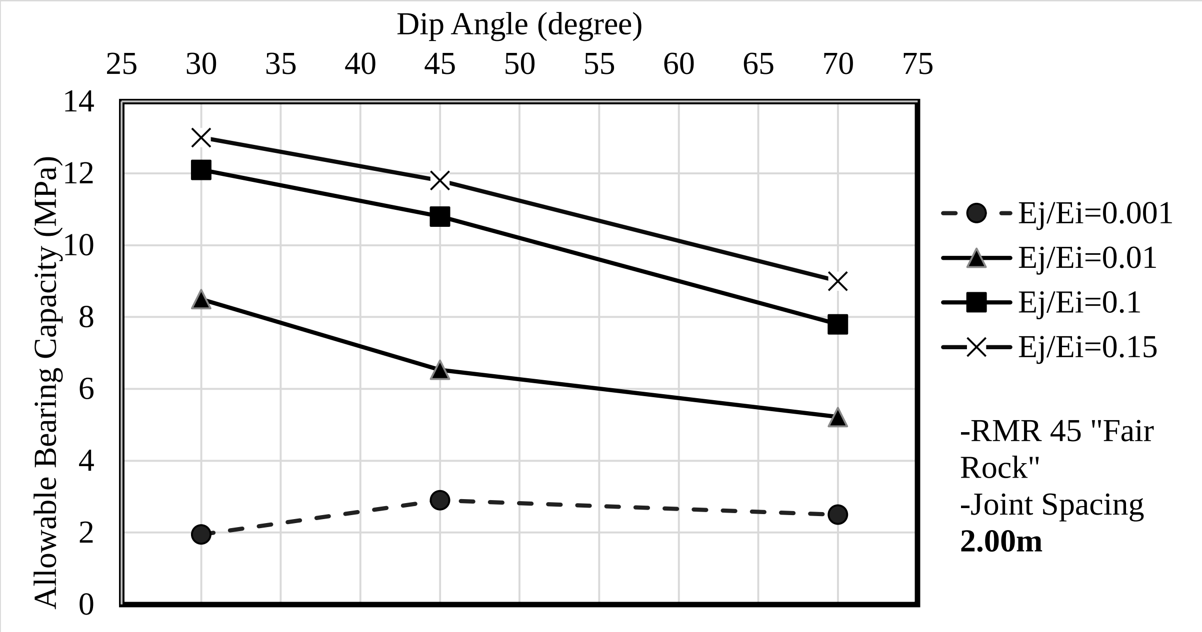


Figure (30): Effect of the joint dip angles on the allowable bearing capacity – RMR 45 – joint spacing 2.00 m

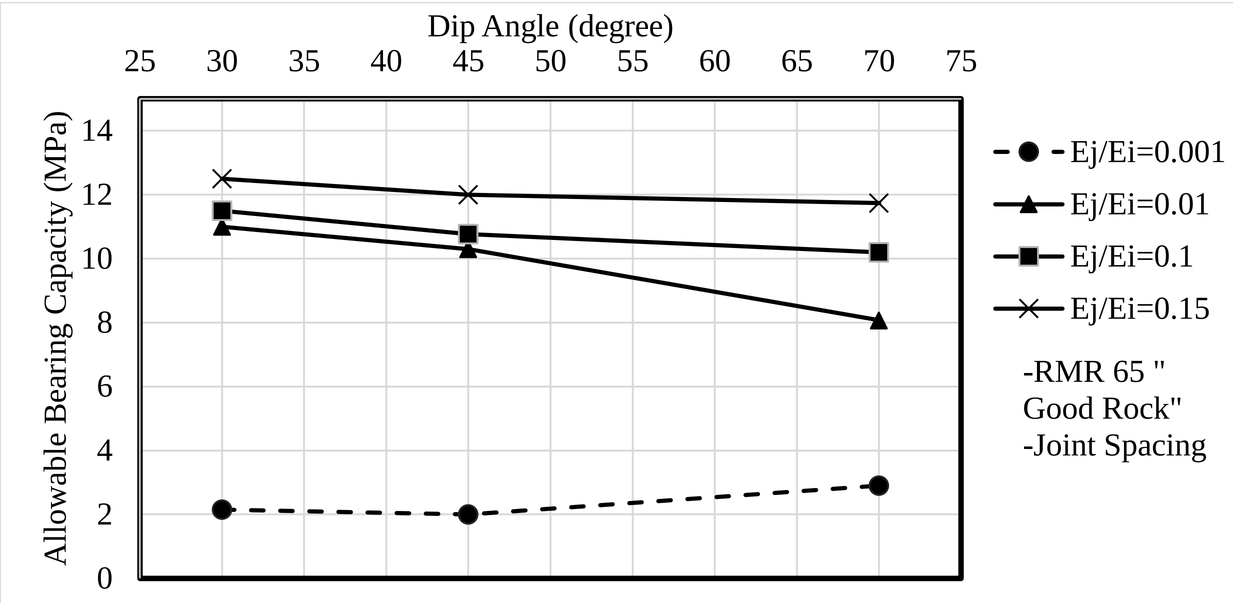


Figure (31): Effect of the joint dip angles on the allowable bearing capacity – RMR 65 – joint spacing 0.40 m

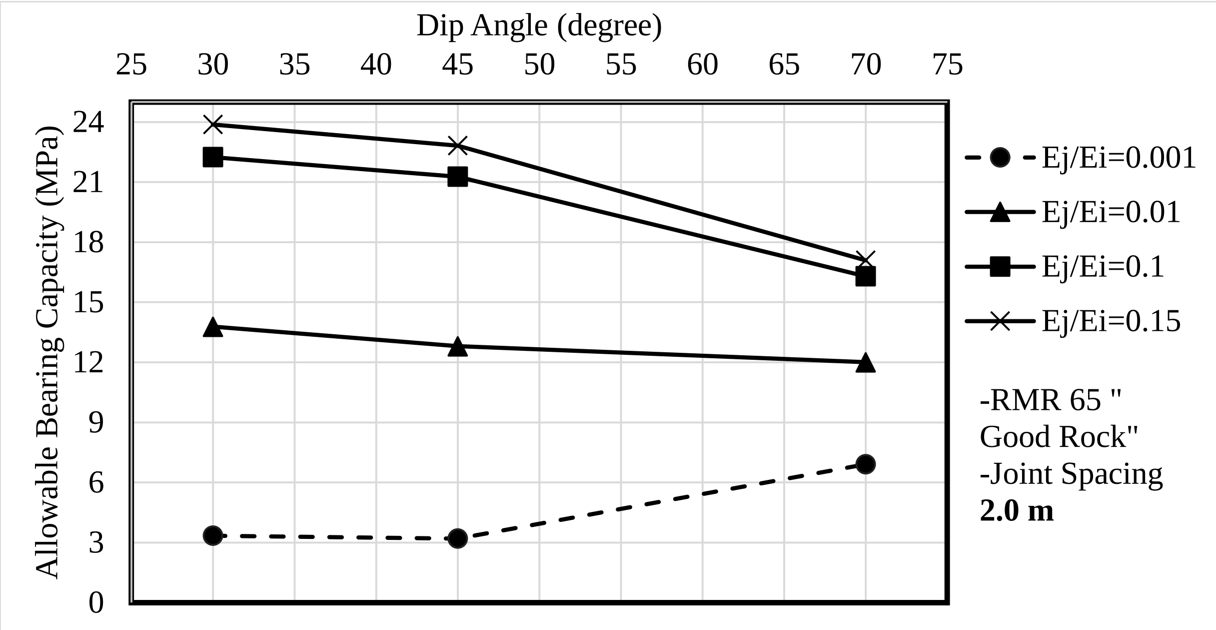


Figure (32): Effect of the joint dip angles on the allowable bearing capacity – RMR 65 – joint spacing 2.00 m

The above figures showed that, as the joint get steeper the allowable bearing capacity reduced. This behavior is repeatable with all the stiffness and cohesion ratios.

At the joint spacing of 0.40 m, the bearing capacity values were nearly in the same range for the ratios 0.15, 0.1, 0.01, in return of the joint spacing 2.00 m the values under the 0.01 ratio diverge from the other two ratios.
It is worth to mention, that the ratio 0.001 in both rock types with both joint spacing pursue the logical behavior nearly in dip angles 30 and 45 degree, but increase in dip angle 70 degree with a sudden unjustified decreasing of the bearing capacity values. This is attributed to the effect of the diminishing the cohesion between the intact block and the joints.

6- Conclusions

The conclusions of the conducted study can be summarized in the following points:

- The finite elements method is a powerful tool to simulate the bearing capacity of the rock mass under loading load.
- Mohr-Coulomb creation can be adapted to simulate the behavior of the rock mass under loading.
- Although the established model on PLAXIS is a detailed discrete way, it is believed that it is the most reliable simulation of the intact and the joints.
- The perpendicular beddings on the loading direction is a strength zone of the mass, continues normal loading on such plane caused closure of the joint, which increase the mass strength.
- The stiffness and the shear strength of both the intact rock and the discontinuities, i.e. Joints are the dominating factors affecting the behavior of the rock mass under loading.
- The orientation of the joints with respect to the loading direction is a significate factor governing the behavior of the joint, i.e. joint normal to loading increase

the strength of the mass by the bedding closure, and the joint inclined with dip angle create a shear zone slides the intact rock blocks along the joint wall.

- The parameters of the discontinuities joints can be a ratio of the intact rock parameters according to the certain study.
- The established model is validated with the conducted results of the case study.
- As the joints get steeper the allowable bearing capacity reduced. This valid with all the stiffness and cohesion ratios except the ratio 0.001, which demolishing the cohesion between the joint and the intact unit.

7- References

A. J. Weltman, J. M. H. (n.d.). Site Investigation Manual. *Construction Industry Research and Information Association, CIRIA, 1983*.

Agharazi, A., Tannant, D. D., & Martin, C. D. (2012). Characterizing rock mass deformation mechanisms during plate load tests at the Bakhtiary dam project. *International Journal of Rock Mechanics and Mining Sciences*, *49*, 1–11. https://doi.org/10.1016/j.ijrmms.2011.10.002

ASTM, D.-95. (2002). Standard test method for unconfined compressive strength of intact rock core specimens. In. American Society for Testing and Materials., D2938-95.

Brinkgreve, R. B. J. (Ed.). (2002). Plaxis: finite element code for soil and rock analyses: 2D-Version 8:[user’s guide]. .

Bieniawski, Z. T. (1989). Engineering rock mass classifications : a complete manual for engineers and geologists in mining, civil, and petroleum engineering. *Wiley-Interscience*, (ISBN 0-471-60172-1), 40–47. https://doi.org/Bieniawski, Z. T. (1989).

Deere D. and Miller R. (1966). *Engineering classification and index properties for intact rock. Tech. Report No AFWL - TR-65-116, Air Force Weapons Lab., Kirtland Air Base, New Mexico.*

Goodman, R. E. (1989). *Introduction to rock mechanics* . New York: Wiley.

Helmy, H., Tawfek, M., & Embaby, K. (2018). Simmulation of Rough Rock Joints under Shear Stress, *8*(7), 70–77. https://doi.org/10.9790/9622-0807047077

Hoek, E. (1994). Strength of rock and rock masses.. *ISRM News Journal*, *2*(2), 4–16.

Kulhawy, F. H., & Goodman, R. E. (1980). Design of foundations on discontinuous rock. . *International Conference on Structural Foundations on Rock*, *1*.

Terzaghi, K. (1943). *Theoretical Soil Mechanics.* . New York: JohnWiley & Sons.