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GEOTECHNICAL CHARACTERISTICS OF THE TERRAIN AND CALCULATION OF BEARING CAPACITY FOR BRIDGE No. 3 OF MOTORWAY ZENICA – SARAJEVO – MOSTAR – BIJAĆA, SECTION INTERCHANGE POČITELJ – ZVIROVIĆI

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ABSTRACT

The paper present a review of the geotechnical characteristics of the terrain at the location of the bridge No. 3 of motorway Zenica – Sarajevo – Mostar - Bijaća, section Počitelj - Zvirovići.

Also, given the suggestion of foundation structures for each column, as well as the calculated bearing capacity.

Key words: *geotechnical properties of soil, foundation, bearing capacity*

INTRODUCTION

As a base for compilation of this Geotechnical Design we used „Study on engineering – geological and geotechnical characteristics of the terrain on location of bridge No. 3“ (compiled by „Geotehnos“ Ltd. Sarajevo, Jun 2014), [1].

As a part of geotechnical explorations the following was done:

- geodetic survey and pegging out of drill hole,
- exploratory drill,
- geological and engineering – geological works,
- laboratory testing.

ENGINEERING – GEOLOGICAL AND GEOTECHNICAL CHARACTERISTICS OF THE TERRAIN AND ROCK

Based on engineering-geological mapping of the terrain and on exploratory drills at the structure location along the expressway route, the following categories are defined:

- Covers /1/,

- Crust abrasions of geological substrate /2/,
- Geological substrate /3/.

Covers are represented by humus clay (1). According to GN 200 that is category II of excavations.

Alluvial deposits include surface blankets of alluvial genetic type:
- pulverulent sandy clay (2b),
- gravel sand (2e),
- muddy sand (2g).

Crust abrasion of geological substrate is represented by degraded horizon limestone (2). According to GN 200 that is category V of excavations.

Geological substrate is represented by:
- limestones (3).

According to GN 200 that is category VI of excavations.

GEOTECHNICAL MODEL OF THE TERRAIN

In order to have adopted the relevant characteristics for materials peel spending geological of substrate horizon of degraded of limestone (2) and geological horizon of limestone substrate (3), made the return analysis in the program RockLab.

The results of analysis are give a reverse in Figs 1 and 2.

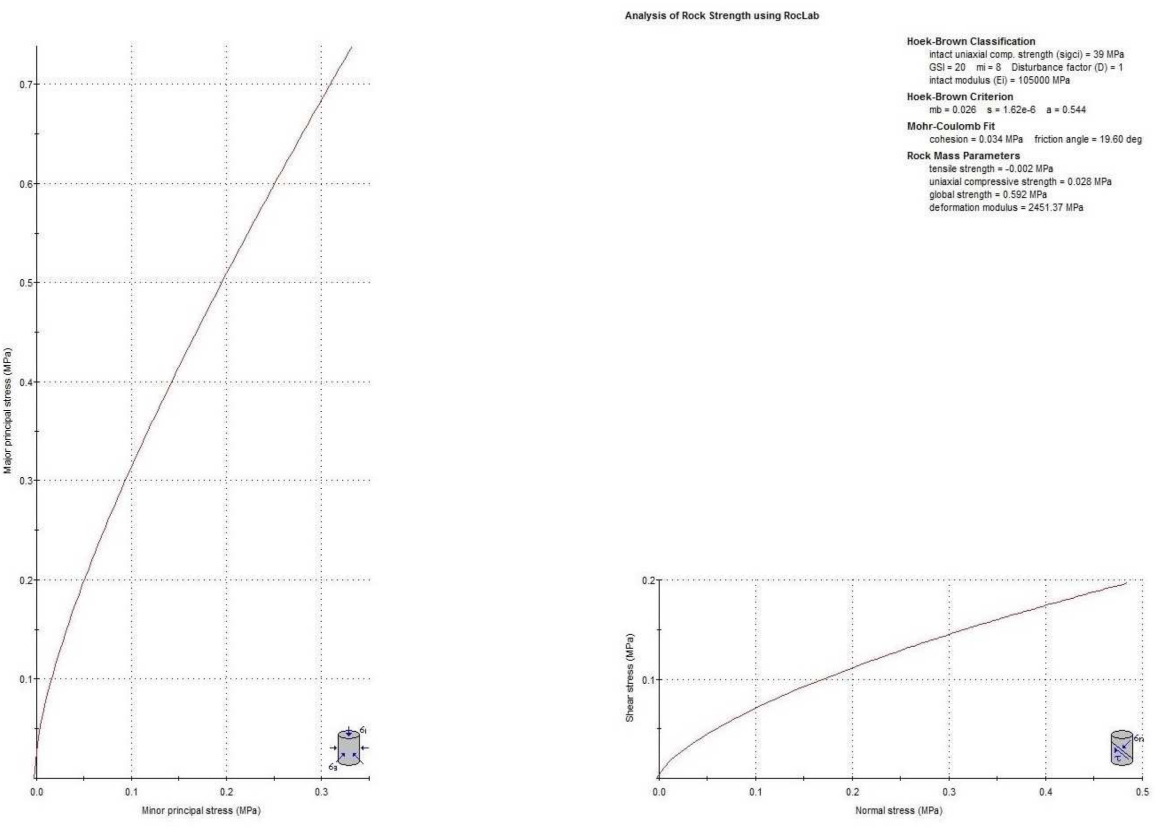


Figure 1 Results of the analysis using software Rock Lab for materials of the crust abrasion of the geological substrate (2)

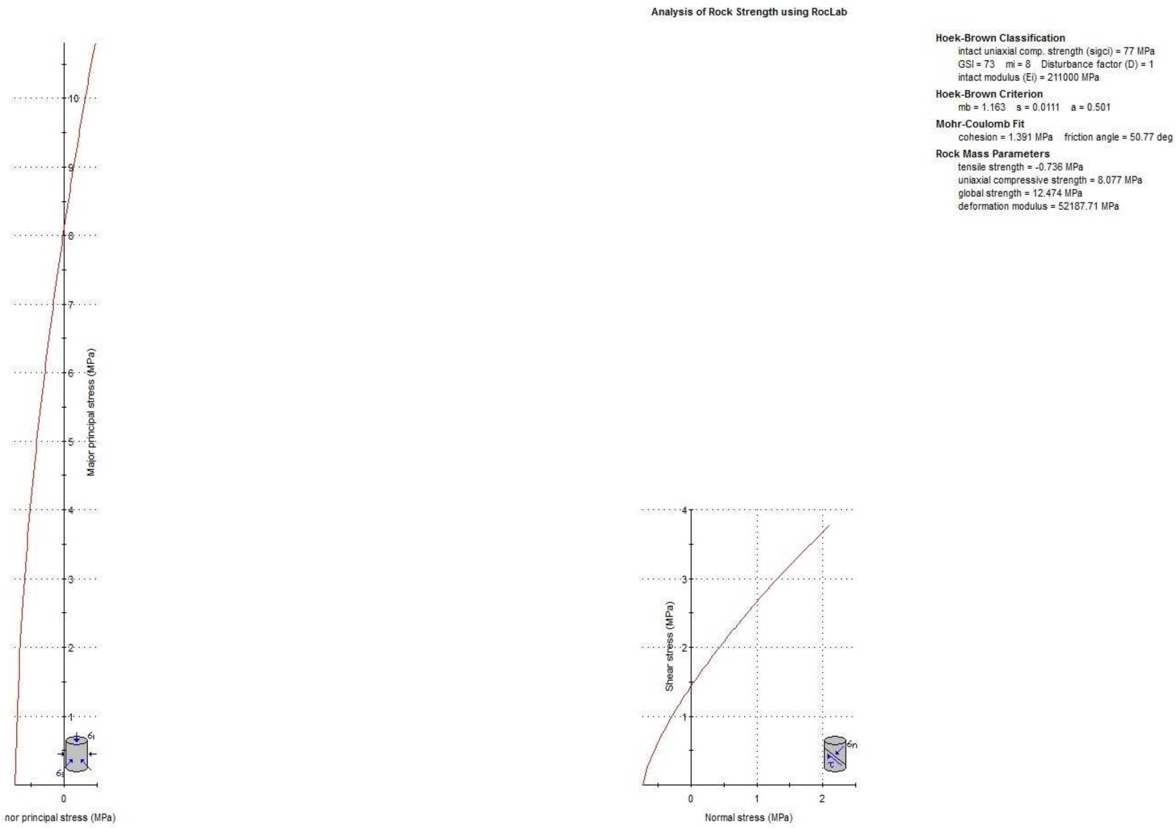


Figure 2 Results of the analysis using software Rock Lab for materials of the geological substrate (3)

Based on terrain and laboratory exploratory works, as well as on engineering – geological determination and classification of exploratory drill core, the following calculation parameters were determined:

for materials crust abrasion of the geological substrate – horizon (2)

- rock mass deformability module $E_s = 1000$ MPa;
- volumetric weight $\gamma = 25$ kN/m^3;
- uniaxial strength $q_u = 39$ MPa;
- GSI 20;
- internal friction angle $\varphi = 20°$;
- cohesion c = 34 kPa.

for materials crust abrasion of the geological substrate – horizon (3)

- rock mass deformability module $E_s = 4000$ MPa;
- volumetric weight $\gamma = 27$ kN/m^3;
- uniaxial strength $q_u = 77$ MPa;
- GSI 73;
- Poisson coefficient $\nu = 0{,}20$;
- internal friction angle $\varphi = 50°$;
- cohesion c = 1391 kPa.

Values of the rock mass deformability module for the materials of geological substrate - horizon (3) were adopted as prescribed by the relevant literature because the analysis done by RockLab yielded high values, [2,3]. Values of the Poisson coefficient yielded by the laboratory testing are somewhat higher than those prescribed by the relevant literature which resulted in said values to be adopted in accordance to the relevant literature as well, [4,5].

FOUNDATION OF THE CONSTRUCTIONS

The structure of the Bridge No. 3 consists of two separate structures: left structure and right structure.

The right structure starts at km 0+294,00 (axis of abutment No 1.), and it ends at chainage km 0+336,00 (axis of column 3.) of the right axis. The left structure starts at km 0+280,00 (axis of abutment No 1.) and it ends at chainage 0+352,00 (axis of column 5) of the left axis. The axes at this stretch of the route do not run parallel to each other.

This was overcome by using RC structures of plate like cross section to be executed in situ. The right structure to be executed with two spans, and the left one with four spans. The right structure spans are of the following static dimensions 21,0 m + 21,0 m, which makes the total length of the structure to be 42,00 m. The left structure is two spans longer, and its static span dimensions are 16,0 m + 2 x 20,0 m + 16,0 m, with total length of 72,00 m.

Abutments are anchored by expansion bearings to the span structure, while middle columns are clamped into the structure.

The columns are to be directly founded over RC footings. Dimensions of the abutments S1 and S3 of the right bridge are 3,50 x 6,50 m, while the dimensions of the column S2 are 5,00 x 5,00 m. Dimensions of the abutments S1 and S5 of the left bridge are 3,50 x 6,50 m, while the dimensions of the columns S2 to S4 are 5,00 x 5,00 m. Thickness of column foundations is 1,50 m.

CALCULATION OF BEARING CAPACITY AND SETTLEMENT UNDERNEATH SHALLOW FOUNDATION FOOTINGS

The calculation was done for the left bridge because the load on its foundations is higher, so it was chosen as more relevant. Calculation of the terrain bearing capacity.

The calculations for the rock mass resistance were done for dimensions, loads and foundation conditions as foreseen by the Design, as well as for adopted parameters of the foundation base strength.

The calculation was done using software Geo5. Image 3 shows the model for calculating shallow foundation bearing capacity using software Geo5. Calculation of foundation footings settling.

Analytic calculation of the settling was done using software GEO 5, which applies the algorithm based on elasticity theory and Boussinesq load distribution. Settling estimation is done based on the premise of concentric or uniformly distributed surface load. Input data used for calculations are intensity and layout dimensions of the load, depth of foundations, compression module, spatial weight and distribution of soil layers.

The settling was calculated applying the following formula:

$$s = \int \frac{d\sigma}{M_k(\sigma)} dz$$

where:
s – settling;
dσ – differential of the additional, actual vertical stress;
$M_k(\sigma)$ – compression module of the foundation soil, dependable on actual vertical stress
z – depth

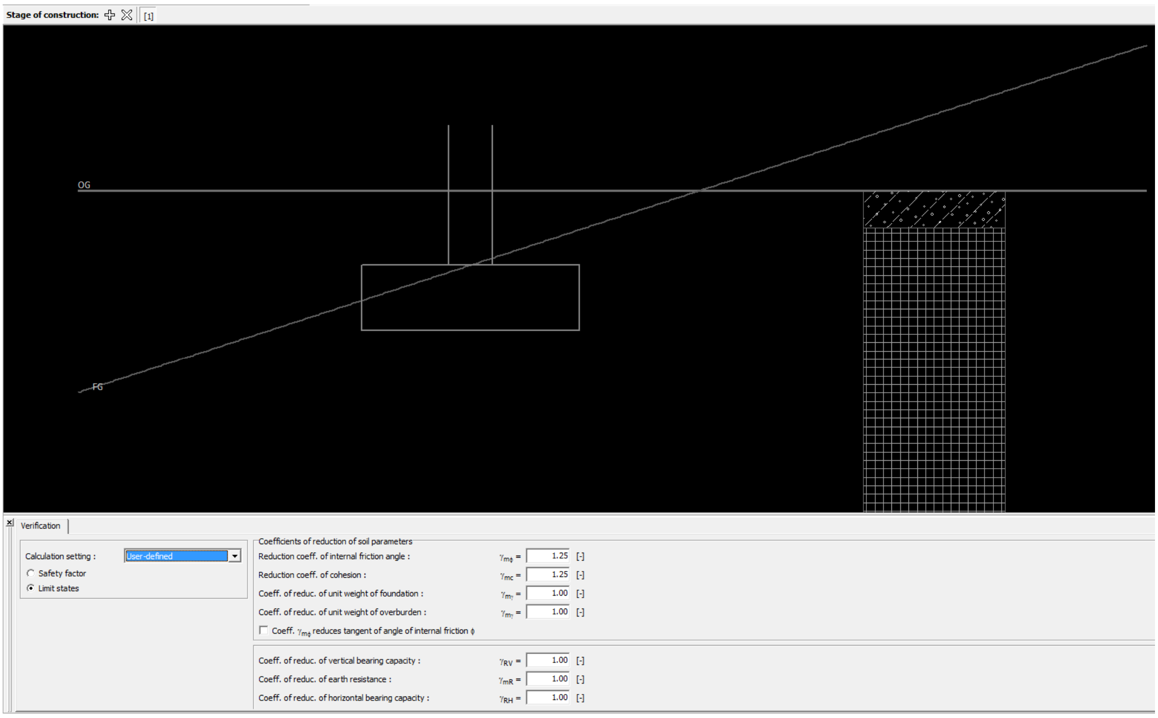


Figure 3 Model for calculating shallow foundation bearing capacity using software Geo5

Additional vertical stress of the soil originating from external load of the rectangular shape was determined by integrating Boussinesq solution for vertical stress in elastic, homogeneous and isotropic interspace, loaded by concentric load on flexible base, Figure 4. Compressibility module is defined as function of actual vertical stress, following the formula:

$$M_k = d\sigma / de = m\,\sigma_R\,(\sigma' / \sigma_R)^{(1-a)}$$

where:

d_σ - differential of actual stress;

d_e - differential of relative vertical deformation;

σ_R - referential actual vertical stress;

σ' - actual stress for which M_k is applicable;

m - characteristic deformation module for σ_R; $m = M_k(\sigma_R) / \sigma_R$;

a - stress exponent.

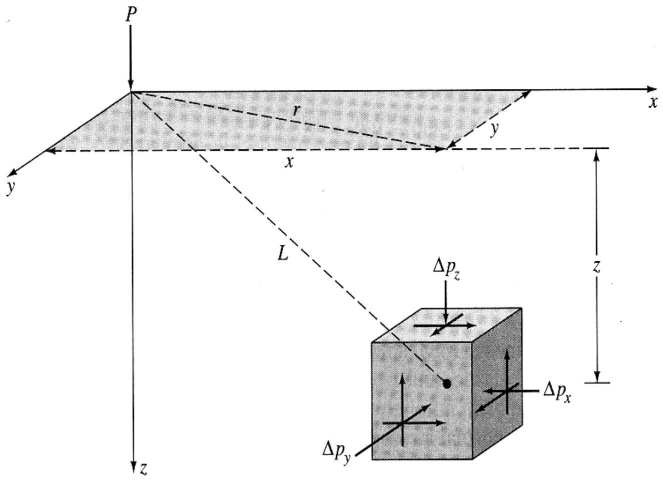
Stress exponent *a* is used to define the measure of compression module increase depending on actual vertical stress; this is the nonlinear element introduced into calculations.

Effects of the upper layers which lay over the foundation reference point was considered as geological load. This calculation did not take into consideration rigidity of the foundation structure so the settling had to be calculated for so called characteristic points, Figure 5. Said points are those in which settling is approximately the same for absolutely rigid and absolutely flexible structure of rectangular layout.

Having in mind that load transfers to the interior of homogeneous interspace, and not to the surface (which is the premise on which algorithm functions), calculated settling is reduced using corrective factor ***k*** as per Fox, which demonstrated the measure of settling for the same soil material parameters when the load is distributed onto the surface and to certain depth within interspace.

Corrective factor as per Fox depends on geometrical relation of the width, length and depth of the foundations.

Calculation is done to the depth at which additional soil stress becomes lesser than selected percentage of geological stress.



$$\sigma_z = \frac{3P\ z^3}{2\pi\ L^5}$$

$$r = \sqrt{x^2 + y^2}$$

$$L = \sqrt{x^2 + y^2 + z^2}$$

Figure 4 Boussinesq solution for vertical stress in elastic interspace

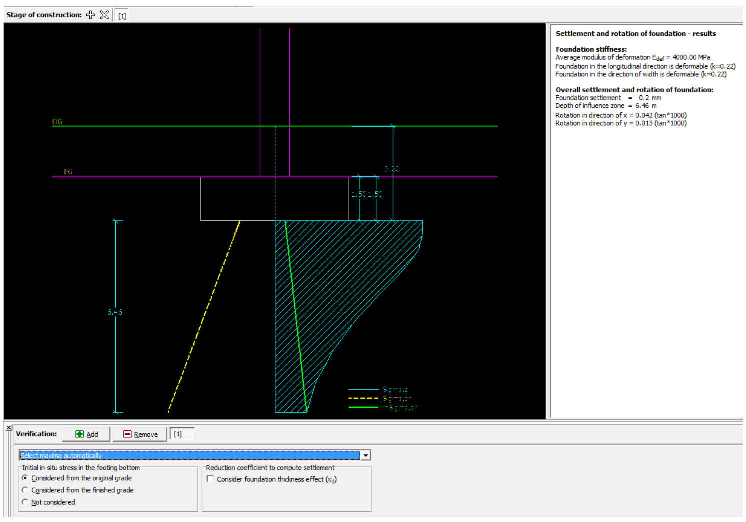


Figure 5 Calculation model for settling of shallow foundation using software Geo5

Settling calculation done applying Janbu theory of tangential module.

Input data and calculation

Input parameters of the load are cross-sectional forces as determined at the joint at the bottom of the column. Calculation took into consideration weight of the footings and embankment above foundations, as well.

Calculation approach PP3 was adopted, i.e. combination of partial factors for limits values of STR and GEO: A1 + M2 + R3. Loads affecting the structure are multiplied by effect factors (γ_F) and effect results (γ_E). Factors of material characteristics were adopted as follows (γ_M): $\gamma_{\varphi'} =$ 1,25 i $\gamma_{c'} =$ 1,25, as well as resistance factors (γ_R) for shallow foundations: $\gamma_{R;v} =$ 1,00 i $\gamma_{R;h} =$ 1,00.

Maximal cross-section forces at the bottom of the columns (excluding the weight of overlay), as yielded by static calculations done in software „RM Bridge", for load combination of ULT (ultimate load, unified calculation and seismic factors), are given in the table 1.

Table 1. Maximal cross-section forces at the bottom of the columns (excluding the weight of overlay), as yielded by static calculations done in software „RM Bridge“, for load combination of ULT

Column	N (kN)	H_x (kN)	H_y (kN)	M_x (kNm)	M_y (kNm)
S2	6926,50	6,82	-50,62	-1185,11	7,32
S2	8248,35	-525,87	128,24	192,67	3592,80
S2	12609,57	71,84	384,05	561,81	-409,86
S2	8681,52	-228,29	-309,89	-4134,45	1767,15
S2	10219,53	-1215,13	-62,89	-1169,09	8680,74
S3	7298,53	-8,38	-23,82	-1084,20	67,32
S3	8600,27	-238,38	108,84	73,51	1783,98
S3	13182,14	-34,70	335,74	594,68	282,32
S3	9208,96	-157,77	-290,52	-4529,04	1393,76
S4	7153,25	-21,48	15,44	-1964,27	53,21
S4	7051,70	724,27	-0,12	-344,55	-3873,25
S4	12181,71	51,07	409,38	-495,74	-357,55
S4	9792,34	2245,03	172,30	1681,52	-12311,85
S4	8380,68	-608,11	-561,06	-6097,85	3177,43

Maximal cross-section forces at the bottom of the columns (excluding the weight of overlay), as yielded by static calculations done in software „RM Bridge“, for load combination of (SLS), are given in the table 2.

Table 2. Maximal cross-section forces at the bottom of the columns (excluding the weight of overlay), as yielded by static calculations done in software „RM Bridge“, for load combination of SLS

Column	N (kN)	H_x (kN)	H_y (kN)	M_x (kNm)	M_y (kNm)
S2	9309,77	68,40	256,03	374,54	-427,43
S2	8107,82	-412,58	132,01	156,63	2747,49
S3	7281,73	-7,36	-15,88	-722,80	58,32
S3	8478,72	-161,25	115,88	41,27	1107,41
S3	9741,54	-35,04	223,82	396,45	295,12
S4	6962,94	-21,42	10,29	-1309,51	54,12
S4	7262,63	446,83	15,17	-412,86	-2292,10
S4	8992,32	72,28	272,92	-330,49	-444,83

In table 3 demonstrates are calculated soil settling bearing capacity and maximal contact stress under foundations, taking load values given in the above table for each column:

From the table above it could be inferred that calculated bearing capacity of the foundation soil is higher than maximal ultimate contact stress, so the settling is within acceptable range (<25 mm as per Rulebook, i.e. <50 mm as per Eurocode 7), which will not cause major changes in distribution of cross-section forces in span structure and bridge columns. In conclusion, proposed dimensions of the foundation footings can be considered to meet the requirements.

Table 3 – Calculated soil settling bearing capacity and maximal contact stress under foundations

Column	Maximal ULS contact stress (kPa)	Calculated bearing capacity of foundation soil R_d (kPa)	Calculated bearing capacity of foundation soil R_d divided by F_m=5 (kPa)	Settling *s* (mm)
S2	750	37853	7570	0,2
S3	540	87361	17472	0,2
S4	1114	36287	7257	0,2

It should be noted that foundation dimensions are somewhat bigger in order to provide for the stability of foundations in relation to possible overturning due to seismic forces.

As values of the permissible bearing capacity yielded by software Geo5 are high, for the safety reasons we undertook calculations using empirical methods.

Goodman Method (1989):

In case of shallow founding in crushed zone where rock mass acts as quasi-continuous geotechnical environment, vertical bearing capacity is calculated using the following formula:

$$q_f = q_u \cdot \left[\tan^2 \left(45 + \frac{\varphi}{2} \right) \right]$$

where q_u represent uniaxial strength of the rock, and φ represents internal friction angle for the crushed rock.

Thus gathered results of the vertical bearing capacity are then divided by global safety coefficient, proposed by Serrano & Olalla, 1998 [6]. This coefficient is determined based on probability of foundation breaking, for rock mass to which Hoek-Brown strength criteria can be applied. The effects of instability caused by change into foundation load were not taken into consideration. Proposed safety factor must include all different forms of instability which are introduced into the calculation of permissible limit values of load bearing capacity:

- static varying of rock mass parameters for which calculation of permissible limit values of load bearing capacity was executed;
- degree to which model of rock mass failure used for calculations corresponds to the actual state.

Global safety factor is expressed as: $F_s = F_p \cdot F_m$

F_m is partial factor which considers possibility of brittle failure. Independent of the foundations size we could take $\sigma_c > 100$ MPa to indicate that the rock mass is brittle in its nature, so the value of F_m ranges from 5-8. With values of $\sigma_c < 12.5$ MPa behaviour of the rock mass during failure can be considered as yielding , so the safety factor considered depends on brittleness.

F_p is partial factor which considers static variability of rock mass parameters: uniaxial rock mass pressure strength, rock mass parameters m_0, and RMR. The following image represents the proposal of the diagram for establishing partial safety factor F_p, Figure 6.

Values adopted for materials of the horizon (2) $F_m = 8{,}0$ and $F_p = 21$, and for materials of the horizon (3) $F_m = 5{,}0$ and $F_p = 36$.

Thus, the values of permissible load bearing capacity for core abrasion of geological substrate amounts to:

$$q_{doz(2)} = 39 \cdot 2{,}04 / 8 / 14 = 0{,}710 \text{ MPa},$$

while for geological substrate this value is:

$$q_{doz(3)} = 77 \cdot 7{,}55 / 5 / 36 = 3{,}229 \text{ MPa}.$$

Eurocode 7 (2008):

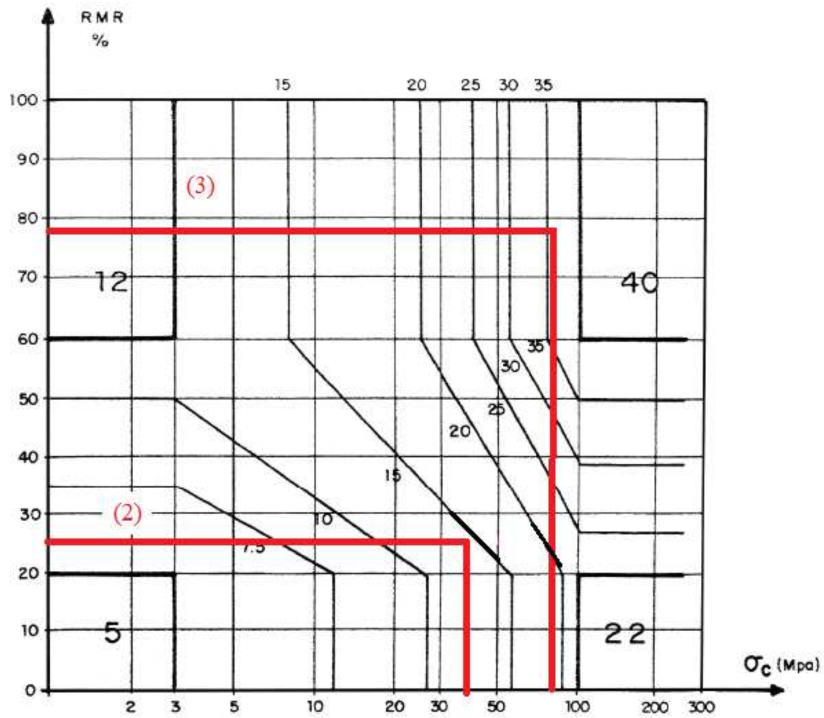


Figure 6. Proposed values of partial safety factor F_p

The estimate of the vertical bearing capacity was done based on BAS EN 19977, [7].

Vertical bearing capacity can be determined from the diagram shown in Image 4, 7. Value determined for the rock mass group 2 thus amounts to 10 MPa, [8].

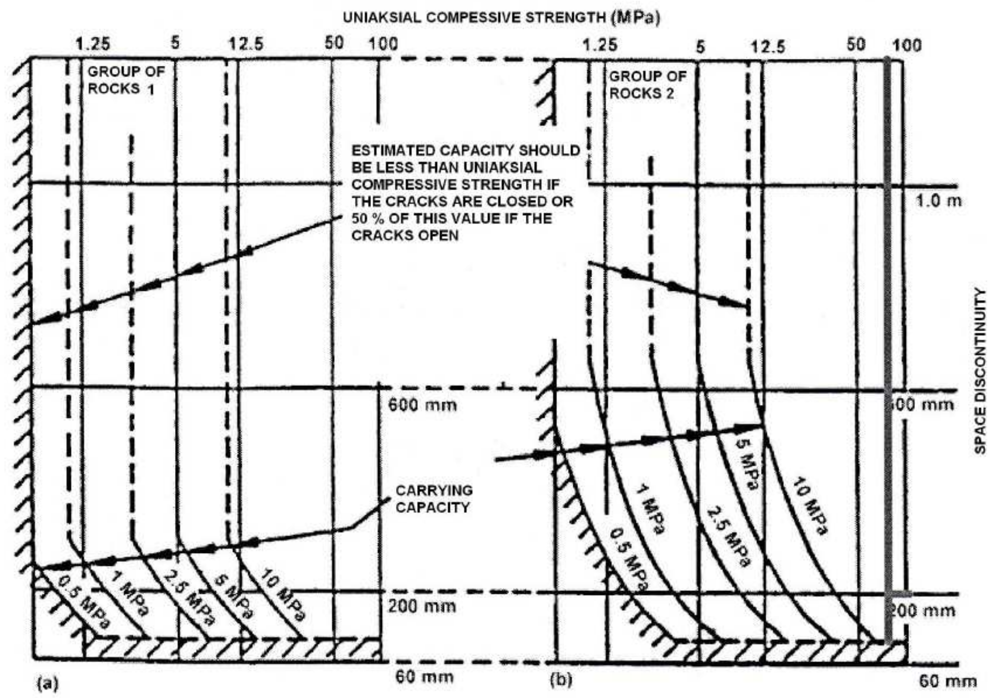


Figure 7. Estimate of the vertical load bearing capacity of square foundation according to BAS EN 1997:2008

If we compare thus determined values of vertical bearing capacity of rock mass (yielded by both software Geo5 and by empirical methods), it can be concluded that vertical loads are lesser than calculated values of vertical bearing capacity of the rock mass.

Table 4. Classification of weak and fractured rock according to Eurocode 7

Group	Rock type
1	Hard limestone and dolomite Carbonated sandstone of lesser porosity
2	Eruptive oolite and marl limestone Well cemented sandstone Hardened carbonated mud Metamorphous rock, including shale and slate
3	Extremely marl limestone Loosely cemented sandstone Slate and shale
4	Non - cemented hardened mud and shale

CONCLUSION

Based on executed field and laboratory testing, as well as on undertake geotechnical analyses for Bridge No. 3 the following can be concluded:

Structure of the Bridge No. 3 consist of two composite structures: left and right one. The right structure starts at km 0+294,00 (axis of abutment No 1.), and it ends at chainage km 0+336,00 (axis of column 3.) of the right axis. The left structure starts at km 0+280,00 (axis of abutment No 1.) and it ends at chainage 0+352,00 (axis of column 5) of the left axis. The axes at this stretch of the route do not run parallel to each other.

The columns are to be directly founded over RC footings. Dimensions of the abutments S1 and S3 of the right bridge are 3,50 x 6,50 m, while the dimensions of the column S2 are 5,00 x 5,00 m. Dimensions of the abutments S1 and S5 of the left bridge are 3,50 x 6,50 m, while the dimensions of the columns S2 to S are 5,00 x 5,00 m. Thickness of column foundations is 1,50 m.

No subterranean waters were detected by undertaken exploratory activities. Designed impact (maximal stress) is lesser than calculated bearing capacity od the foundation soil. If we compare thus determined values of vertical bearing capacity of rock mass (yielded by both software Geo5 and by empirical methods), it can be concluded that vertical loads are lesser than calculated values of vertical bearing capacity of the rock mass. For said founding conditions and designed loads, settlement of the foundation structure could be expected ranging up to 0,2 mm.

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LITERATURE

[1] Study on engineering – geological and geotechnical characteristics of the terrain, mission G21. „Geotehnos“ Ltd. Sarajevo, Jun 2014.
[2] Obradović, R. & Najdanović, N. (1999). Mehanika tla u inženjerskoj praksi. Rudarski Institut Beograd, R Srbija.
[3] Stević, M. (1991). Mehanika tla i stijena. Rudarsko–geološki fakultet, Tuzla
[4] Ćerimagić, Đ. (2009). Inženjerska geologija. Građevinski fakultet u Sarajevu.
[5] Đurić, N. (2011). Hidrogeološka i inženjerskogeološka istraživanja. Građevinski fakultet Subotica, Tehnički institut Bijeljina, Subotica, Bijeljina.
[6] Serrano A. & Olalla C. (1998).Ultimate bearing capacity of an anisotropic discontinuous rock mass, part I: basic modes of failure. Int J Rock Mech Min Sci 1998;35(3): 301–24.
[5] BAS EN 1997-1: Eurokod 7 – Designing geotechnical structures - Volume 1: General rules.
[6] Miščević P. & Števanić D. (2006). Granična nosivost plitkih temelja na stijenskoj masi. Građevinarbr. 58, Zagreb, R Hrvatska.