



## Design of reinforced gabion wall: example of retaining wall for "Vidikovac" rest area on the island of Brač

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**Abstract:** For purpose of constructing the "Vidikovac" rest area on the state road D115 Humac - Bol, not far from town of Bol on the island of Brač, a 9m retaining wall was necessary to form a plateau. Since the denivelation could not be formed as cascading, due to the request space for rest area, options for the construction of a 160 m long classically reinforced concrete wall and a gabion wall were investigated. The gabion wall with reinforced backfill was selected. Calculation sequence, as well as advantages and disadvantages of this type of construction are presented in this paper.

**Key words:** gabion, reinforced backfill, retaining wall, design, rest area

### Proračun gabionskog zida s armiranim zasipom na primjeru potporne konstrukcije za potrebe odmorišta „Vidikovac“ na otoku Braču

**Sažetak:** Za potrebe izgradnje odmorišta „Vidikovac“ uz državnu cestu D115 Humac – Bol, nedaleko od Bola na otoku Braču, ukazala se potreba za savladavanje denivelacije ukupne visine od 9 m. Zbog malog raspoloživog prostora uvjetovanog imovinskih odnosima, te zbog zahtjeva uporabnog prostora odmorišta, razmotrene su opcije izgradnje klasičnog armirano-betonskog zida i gabionskog zida ukupne duljine 160 m. U konačnici je odabrana izgradnja gabionskog zida s armiranim zasipom, te su slijed proračuna, prednosti i nedostaci ovog tipa konstrukcije prikazani u ovom radu.

**Ključne riječi:** gabion, armirani zasip, potporni zid, proračun, vidikovac



## 1. INTRODUCTION

A proposal for type of retaining structure with an appropriate geotechnical calculation was requested for construction of the rest area on a part of the state road D115, section 1, Gornji Humac - Bol at chainage km 7+750 near Bol on the island of Brač. Since the rest area would normally be used by vehicles leaving Bol and continuing their journey to Gornji Humac and further to the ferry ports in Supetar or Sumartin, the conceptual solution specifies construction of a longitudinal rest area that follows the state road in around 160 m (Figure 1).



Figure 1. Position of the rest area on D115

Connection to the planned rest area is "right - right," which means that entry is possible only from the west direction (Bol) and exit to the east direction (Gornji Humac). Access to the plateau from the state road is realized over the exit lane, the so-called "wedge-like outlet" of the total length of 40 m, while entry to the state road for vehicles from the rest area is provided directly by applying the connection exit radius  $R=20$  meters. The passage of all emergency vehicles (fire brigade, ambulance, etc.) and buses is uninterrupted.

Considering the lay of the land, following preliminary analysis of construction costs of a conventional reinforced-concrete wall and a gabion wall, construction of a gabion wall with reinforced backfill on the south side of the rest area was selected. For simplicity of construction, and due to the need to reinforce the backfill because of the significant height of the structure, the wall is made of standard elements of reinforced Terramesh system with height varying from 1.0 to 9.0 m, since they include gabion and tied reinforcement made of galvanized plasticized wire. [1]

### 1.1 Geotechnical properties of the site

According to the geological map (Jelsa 1:100 000), the subject site is made of limestone with dolomite lenses ( $^3K^3_2$ ) and stratified limestone ( $K_2^2$ ) in alternation (Figure 2). [2] Their occurrence is confirmed on rock mass outcrops in the field, as well as in side cuts at the site. The underlying rock mass occurs on the ground surface at the location or is partially covered on the surface with a layer of dark brown to reddish-brown dusty clay with fragments of parent rock debris. The initial part of this layer up to 20-50 cm in depth is organically contaminated in some places (plant roots).

On the roadside slope (on the south side), at the site planned for construction of the widening for the lookout plateau, there is a 0.5 - 2.0 m thick embankment on the surface. The embankment is made of stone material from excavation of the road side cut. The embankment contains fine gravel and large stone blocks.

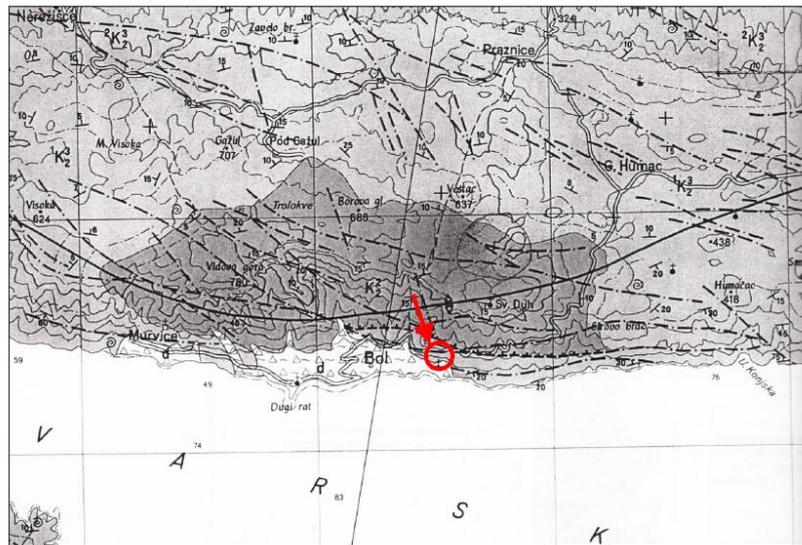


Figure 2. Geological map of the wider area of Bol (the island of Brač)

The rock mass is markedly weathered and cracked by several fracture systems in the top part up to about 2.0 - 3.0 m in depth. Strata joints are spaced from several centimetres to several tens of centimetres, with dip about  $40^{\circ}$ - $50^{\circ}$ , with strike approximately north-northeast. Strata joints are mostly closed, while apertures of other cracks are from several millimetres to several centimetres. They are typically filled with medium to highly plastic, clayey silt brown-red to dark brown in color. Crack walls are not weathered, rough and coated with an orange-brown calcitic coating.

Two secondary fracture systems are determined on the side cut along the existing road. One with dip of about  $60^{\circ}$ - $70^{\circ}$  and strike approximately west, and the other with sub vertical dip and strike approximately east-west. Due to karstification of the rock mass, it was not possible to confirm regularity of the secondary fractures on the entire site before excavating foundations of the structure (and so inspection of the excavation was subsequently performed).

With increasing depth (at depths of 2.0 - 3.0 m from the initial ground surface), fracturing of the rock mass is generally lower, fractures are narrower, and the rock mass is stronger and more compact. In hydrogeological terms, fractured and karstified limestone deposits have a fracture and possible cavernous porosity, and filter precipitation water into the underground relatively fast. For this reason it is necessary to take into account protection of the underground from pollution.

According to the Seismological Map (HRN EN 1998-1: 2011/NA:2011, State Geodetic Administration - Republic of Croatia - map of earthquake areas), the investigated site is situated in an area with an estimated maximum horizontal acceleration of 0.21 g. For seismic calculation of the structure, when determining local soil conditions, consider that the soil class is A (limestone rock mass). [3]

## 2. SELECTION OF THE STRUCTURE

The selected retaining structure is soil reinforced with Terramesh system (mesh type 8 x 10 cm, wire diameter 2.7 mm, GALMAC + PVC coating) [4]. The Terramesh system consists of gabions and a tied soil reinforcing "tail". Both parts are made of galvanized and PVC protected wire. The selected system dimensions are 6 x 2 x 1.0 m and 4 or 5 x 2 x 1.0 m up to a height of 6.0 m from the upper part of the structure, or the first six layers of the structure.



The dimensions 4 x 2 x 0.5 m were selected from the seventh to the twelfth layer, or for the heights from 6 to 9 m measured from the top of the structure (Figure 3).

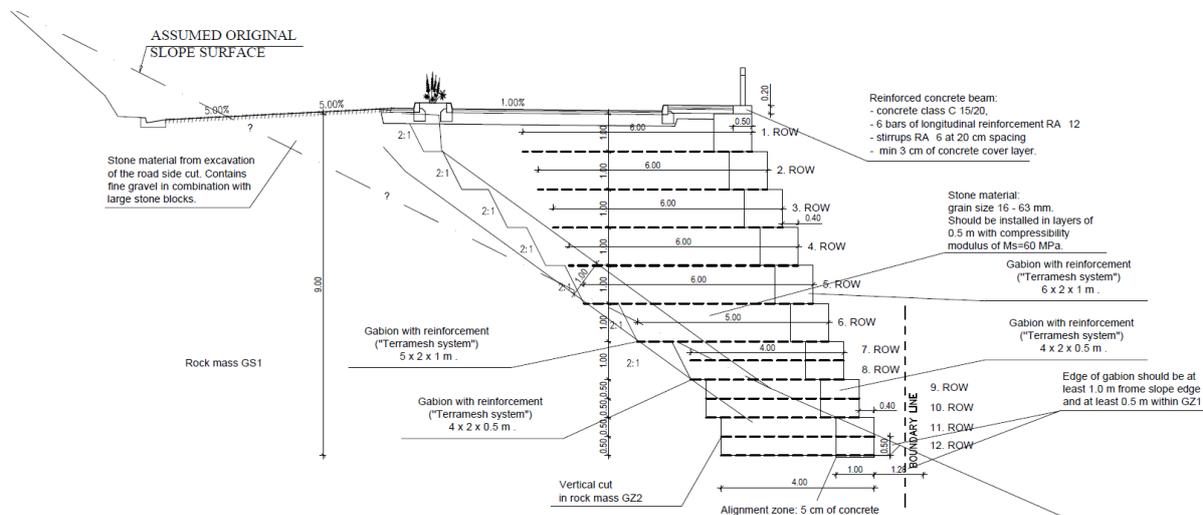


Figure 3. Normal cross section of the retaining structure

Gabions with dimensions of 1 x 1 x 2 m (up to a protection height of 3.0 m) are placed in the initial part of the structure (connection of the rest area to the existing road). The gabion construction mesh is made of galvanized wire with a diameter not less than 3.0 mm and a mesh aperture not greater than 8 x 10 cm. The backfill is made of stone material with grain size 16-63 mm. The stone material is made of crushed stone of sharp-edged grains in order to be able to achieve the required characteristic internal friction angle of at least  $\phi_k = 35^\circ$ . Generally, stone material for which it cannot be determined that it has the required internal friction must not be used. The embankment is made in 0.5 m thick layers, with compaction of each layer with vibratory tampers. Each layer is compacted to a modulus of compressibility of  $M_S = 60$  MPa, which is determined for each layer by the plate load test.

The foundation surface of the retaining structure longitudinally follows the gradient of road level line (the gradient of 5%). The foundation surface of the retaining structure is formed by cutting into the slope on the south side of the existing road. Part of this slope is formed as embankment constructed for the existing road, and part is a natural slope in rock mass. The rock mass was cut for foundations of the structure, i.e. the entire foundation surface of the lowest layer of the Terramesh system and gabions at the beginning of the structure is on the rock mass, specifically so that the outer edge of the system and gabions is at least 1.0 m away from the edge of the slope and at least 0.5 m "inside" the rock mass. The foundation level is determined as the bottom of the lowest layer lowered by 5 cm. These 5 cm is the rock excavation levelling zone, which is filled with blinding concrete, but only in the zone under the gabion part.

Side cutting into the slope required for construction of the support structure was carried out in a cascading manner. The height of an individual cascade is 1.0 m (Figure 3). Each cascade was cut into the rock mass vertically, and into the existing road embankment at a gradient 2:1. For placement next to the existing road embankment, cutting was carried out by placing mesh ends - "tails" of the Terramesh system at least 1.0 m inside the existing embankment, i.e. the surface layer of the existing embankment was removed in a thickness of 1.0 m from the previous slope surface. In this way, better bonding of the new and existing embankment has been achieved. To achieve the best possible anchoring of the tail of the Terramesh system, the mesh is at least 2.0 m long from the gabion front face edge. End of the mesh extends to the edge of the cascading side cut, so that the end of the mesh itself is on the rock mass in the part where rock mass is reached by cutting.



When constructing the gabion part (front face of the structure), the fill is made of stone material 60-80 mm in diameter, and the front face of gabions is made of manually arranged larger crushed stones to achieve a dry stone wall texture typical of this area (Figure 4). After completing the filling, the baskets are closed with lids, and are bound with vertical and angle ties. The procedure was successively repeated in the same manner in subsequent layers.



Figure 4. As-built condition

### 3. CALCULATION METHODOLOGY

Stability calculations of the retaining structure on slope were conducted based on available data on slope geometry and geological structure of the terrain. The calculations were carried out according to the standards HRN EN 1997-1:2012, HRN EN 1997-1:2012/A1:2014, HRN EN 1997-1:2012/NA:2012 (Eurocode 7 - Geotechnical design + National Annex). [5]

The seismic calculation was conducted according to the standard HRN EN 1998-5:2011 (Eurocode 8 - Design of structures for earthquake resistance; Foundations, retaining structures and geotechnical aspects). [6]

According to HRN EN 1997-1:2012 (Eurocode 7), all geotechnical structures are treated as permanent so that the effect of seismic action will be included in all calculations. The calculation was made for the return period of 475 years.

According to HRN EN 1997-1:2012/NA:2012, this geotechnical structure belongs to the geotechnical category of construction GC2 (excavations, anchors, usual types of construction and foundations) for which geotechnical investigations include routine investigations, including boring, field and laboratory tests, while calculation procedures comprise routine stabilization and deformation calculations based on design procedures from EC7. The design approach DA3 was used, and the calculation values of materials for ultimate limit state analyses were derived from characteristic values with application of partial coefficients

The stability calculation was carried out for the most critical combination of loads:

I - exceptional situation: own weight + earthquake + traffic load



### 3.1 Traffic load

According to HRN EN 1991-2:2012 [7] (actions on structures/ traffic loads of bridges) loads acting on traffic areas should be according to Figure 5. The first 3 m wide traffic lane is loaded with two axle loads  $Q_{ik} = 300$  kN and a continuous load  $q_{ik} = 9$  kN/m<sup>2</sup>, where the concentrated load is replaced with a uniform load distributed on the vehicle area  $3 \times 5 = 15$  m<sup>2</sup>. According to the expert report DIN Fachbericht 101 [8], the correction factor of 0.8 is recommended for the concentrated load. This gives:

$$q_k = 0.8 \cdot (2 \times 300 / 15) = 32 \text{ kN/m}^2 - \text{substitute uniform distributed load}$$

$$q_{ik} = 9 \text{ kN/m}^2 - \text{continuous load}$$

$$p_{k1} = 32 + 9 = 41.0 \text{ kN/m}^2 - \text{total continuous load}$$

$$p_{d1} = p_{k1} \cdot \gamma_Q = 41 \cdot 1.5 = 61.5 \text{ kN/m}^2 - \text{calculation continuous load for DA3}$$

The other 3 m wide traffic lane is loaded with two axle loads  $Q_{ik} = 200$  kN and a continuous load  $q_{ik} = 2.5$  kN/m<sup>2</sup>. This gives:

$$q_k = 0.8 \cdot (2 \times 200 / 15) = 21.4 \text{ kN/m}^2 - \text{substitute uniform distributed load}$$

$$q_{ik} = 2.5 \text{ kN/m}^2 - \text{continuous load}$$

$$p_{k2} = 21.4 + 2.5 = 23.9 \text{ kN/m}^2 - \text{total continuous load}$$

$$p_{d2} = p_{k2} \cdot \gamma_Q = 23.9 \cdot 1.5 = 35.9 \text{ kN/m}^2 - \text{calculation continuous load for DA3}$$

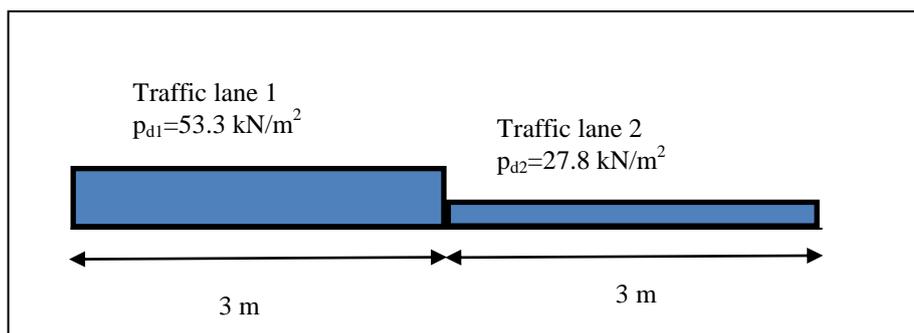


Figure 5. Sketch of traffic load

In the calculation model (software suite Rocscience Slide, [9]) all loads are validated with the associated partial safety factor. For this reason, characteristic values of loads were input into the model.

### 3.2 Seismic load

The subject location is situated in an area with estimated maximum horizontal acceleration of  $\alpha = 0.21$  g, with remark that this is the soil class A (limestone rock mass). For pseudostatic methods, inertia forces in the horizontal direction on each part of soil mass are calculated according to (1):

$$F_H = 0.5 \cdot \alpha \cdot S \cdot W \quad (1)$$

According to HRN EN 1998-1:2011/NA:2011 [3], Type 1 elastic spectrum is used for determining S, so  $S=1.0$  is used for the soil type A. Consequently, calculation of inertia forces is conducted according to (2):

$$F_H = 0.5 \cdot 0.21 \cdot 1 \cdot W = 0.11 \cdot W \quad (2)$$



#### 4. DEFINING STRENGTH PARAMETERS

Two geotechnical zones are defined on the observed slope, which are included in the numerical calculation. These are the bedrock zone (geotechnical zone 1 - GZ1) and the stone material backfill zone (geotechnical zone 2 - GZ2).

##### 4.1. Defining the strength parameters of underlying rock mass (GZ1)

The rock mass strength parameters were calculated using the theory of empirical strength criterion according to Hoek & Brown and equivalent Mohr-Coulomb failure criterion, based on uniaxial strength (GSI - Geological Strength Index). The value of geological strength index GSI for the rock mass at the subject location was determined based on results obtained by geological mapping and empirical data.

The classification parameters and corresponding rating value (RMR) for GZ1 are presented in Table 1. Calculation of rock mass parameters is performed using the RocLab software [10]. The calculation sequence and results are presented in Table 2.

Table 1. Classification parameters and corresponding rating value (RMR) for GZ1

Parameter	Value range	Points	
Uniaxial compressive strength	40 MPa		5
Rock quality designation RQD	25% - 50%		8
Spacing of discontinuities	0.06 -0.2 cm		8
Condition of discontinuities			
- discontinuity persistence	> 20 m	0	
- aperture	0.1 - 1.0 mm	4	
- roughness	rough	5	
- infilling	hard <5mm	5	
- weathering	slightly weathered	5	
Groundwater conditions	damp		8
Adjustment for joint orientation (favorable)			-5

For the observed rock mass → RMR=43 points → GSI=RMR-5=43-5=38

Table 2. Calculation of rock mass parameters [10].

Hoek-Brown criterion	Determination of the failure envelope range	Rock mass parameters
GSI = 38	$\sigma_{3,max} = 0.46$ MPa	$\sigma_c = 1.422$ MPa
$\sigma_{ci} = 40$ MPa	Application: slopes	$\sigma_t = 0.061$ MPa
D = 0.1	$\gamma = 0.026$ MN/m <sup>3</sup>	$\sigma_{cm} = 5.063$ MPa
mi = 8	Slope height: 20 m	$E_{rm} = 6084.44$ MPa
<b>Hoek-Brown parameters</b>	<b>Mohr-Coulomb parameters</b>	
s = 0.00143	c= 0.272 MPa	
$m_b = 0.938$	$\varphi = 50.07^\circ$	
a = 0.509		

For global stability calculation purposes, the parameters of cohesion and internal friction angle of the rock mass are selected:  $c'_k = 270$  kPa,  $\varphi'_k = 50^\circ$  and  $\gamma = 26$  kN/m<sup>3</sup>.



## 4.2. Defining embankment strength parameters (GZ 2)

A zone of fill material is present in the upper part of the road. For global stability calculation purposes, the parameters of cohesion and internal friction angle of that layer are selected [11] as following:  $c'_k=1$  kPa,  $\varphi'_k=35^\circ$  and  $\gamma=20$  kN/m<sup>3</sup>.

For global stability calculation purposes, the parameters of cohesion and internal friction angle of the embankment of stone material are selected:  $c'_k=0$  kPa,  $\varphi'_k=35^\circ$  and  $\gamma=20$  kN/m<sup>3</sup>.

## 5. THE NECESSARY ANCHORING LENGTH AND TENSILE STRENGTH OF BACKFILL REINFORCEMENT

According to the normal cross section (Figure 3), determination of properties for embankment of reinforced soil, required for selection and installation of the Terramesh system, anchoring lengths and tensile resistance is given below.

- (i) INPUT DATA for reinforced soil embankment  
 $\gamma = 20.0$  kN/m<sup>3</sup>;  $c = c_k = 0$  kPa;  
 $\varphi = \varphi_k = 35^\circ$ ;  $\delta \approx \varphi_k = 35^\circ$  (interlocking in the mesh) ;  
 $K_A = \text{tg}^2(45^\circ - \varphi_k/2) = 0.27$
- (ii) CALCULATION FOR THE WALL OF HEIGHT  $H = 9$  m, made of 6 layers 1 m each + six layers 0.5 m each

According to the manufacturer's specification [4] the design tensile strength of the Terramesh system mesh is  $T_{v,d} = 26$  kN/m'. Load taken by the reinforcement of each layer must be lower than the design load. The tensile force in each reinforcement layer is determined according to:

$$T_i = (z_i + z_{i-1}) \cdot 0.5 \cdot \gamma \cdot K_A \cdot (z_i - z_{i-1}) \quad (3)$$

and calculation for all heights is presented in Table 3.

Table 3. Tensile force in each reinforcement layer

z (m)	1.0	2.0	3.0	4.0	5.0	6.0	6.5	7.0	7.5	8.0	8.5	9.0
$T_i$ (kN/m')	2.70	8.10	13.50	18.90	23.30	28.70*	16.88	18.23	19.58	20.93	22.28	23.63

\* - the value is accepted because the difference up to the design bearing capacity is 10%, which can be covered by the safety factor, and the load is calculated for conservatively assumed backfill parameters.

The greatest load occurs in the lowest layer and for the 0.5 m high gabion it is:

$$T = H_{max} \cdot \gamma \cdot K_A \cdot h_i = 8.75 \cdot 20 \cdot 0.27 \cdot 0.5 = 23.6 \text{ kN/m}' < T_{v,d} = 26 \text{ kN/m}' \quad (4)$$

Reinforcement anchoring length beyond the failure surface/active wedge -  $l_{ei}$ , along the upper edge of the layer  $i$ , required to hold the force  $T$  by the resulting mutual friction  $2 \cdot N \cdot \tan \delta$ :

$$l_{ei} \geq \frac{T \cdot \gamma_{G,dst}}{2N \cdot \tan \delta \cdot \gamma_{G,stb} \cdot \frac{1}{\gamma_{R,h}}} = \frac{23.6 \cdot 1.0}{2 \cdot 9 \cdot 20 \cdot \text{tg} 35^\circ} = 0.09 \text{ m} \quad (5)$$

- (iii) TOTAL LENGTH OF REINFORCEMENT (geogrid)  
 selected:  $l_{ui} = 2.0$  m in all sections,  $i = 1 - 11$



## 6. GLOBAL STABILITY CALCULATION

Stability analysis of the retaining structure was carried out using the software suite Rocscience Slide 7.0 [9] on the model shown in Figure 6. Circular failure is assumed in the calculation, and also within the limit equilibrium method (Bishop method). The calculation was carried out with automatic search for critical slip surface (with smallest factor of safety) for defined boundary conditions.

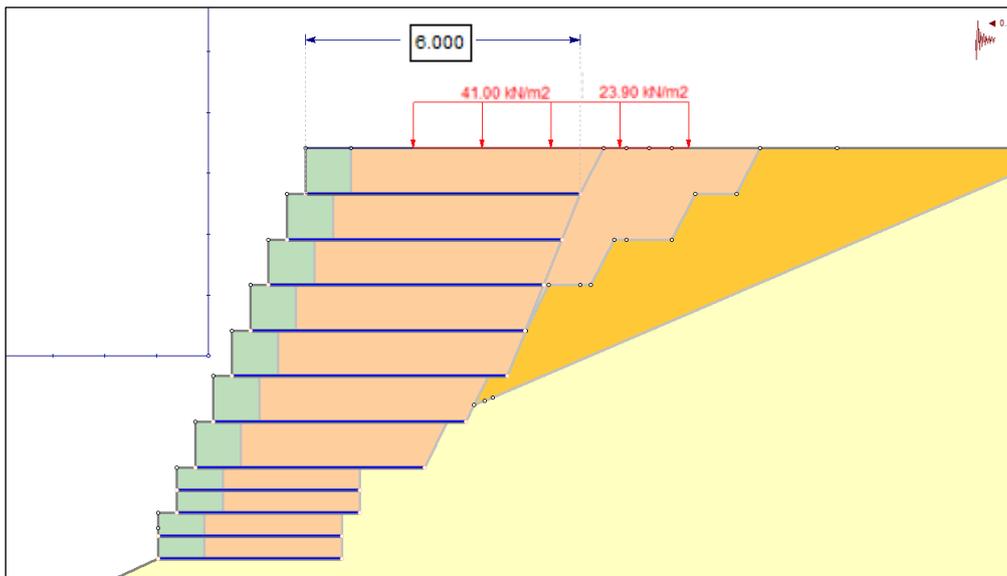


Figure 6. Geometry of the reinforced soil embankment structure used in the global stability analysis model

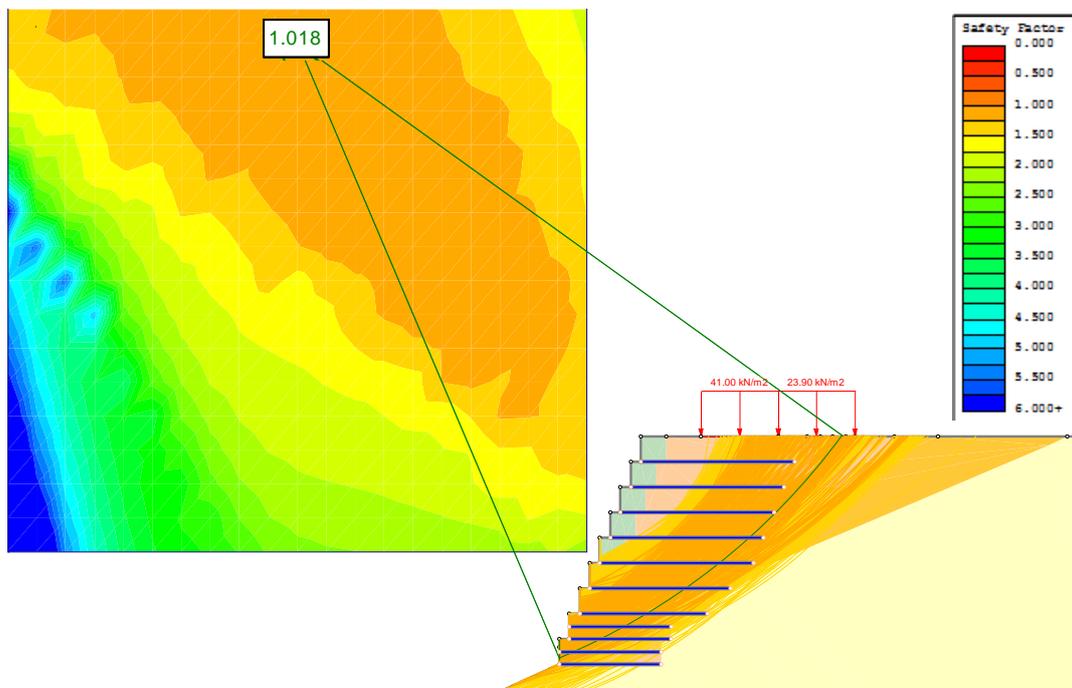


Figure 7. Stability control of the reinforced soil embankment structure with Terramesh system (critical load combination: own weight + traffic + earthquake)



Calculation values of cohesion and internal friction angle were used in the analyses, reducing the characteristic values of soil parameters according to DA3.

Two approaches are used to define stable side cuts and cuts in rocks. According to the first approach, potential failures are analysed by mapped fracture systems. According to the second approach, rock is observed as a quasi-homogenous environment with strength parameters dependent on the type of rock and condition of rock mass as well as normal stress.

The safety factor  $F_s=1.018 > F_{smin}=1.0$  is obtained by global stability analysis (Figure 7) for the exceptional load situation (own weight + traffic load + earthquake), therefore the stability condition is satisfied.

## 7. CONCLUSION

This paper presents the main considerations and the sequence of calculation of the gabion wall with reinforced backfill. Although reinforced concrete retaining structures are still preferred in the Dalmatia region, in this example, by selecting the gabion wall and comparing it with the considered concrete wall, the following advantages are observed:

- selected solution is 30% cheaper than a reinforced concrete wall of the same height,
- construction is less demanding (everything is made with "dry procedure"),
- it is not necessary to construct a drainage system because the wall itself is permeable,
- lower additional load on the slope due to the lower weight of the reinforced soil structure,
- sliding resistance of the slope itself (underlying base) is greater.

In the context of Dalmatian landscape with traditional dry stone wall construction [12], it is worthy to emphasize the autochthonousness and excellent integration into the environment of the Mediterranean landscape.

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