

Behavior category and design loads for conventionally excavated tunnels

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ABSTRACT: The geostatic behavior of tunnel sections may be significantly different even when the sections are excavated in a medium characterized by identical geotechnical properties. In fact, several other factors have an important impact in the response: sometimes, the in-situ natural stresses of the medium in which the excavation is performed and some other times, the geometric and topographic characteristics of the surface. For intermediate tunnel depths, the geo-mechanic behavior of the tunnel, and as a consequence, the loads imposed on the support and the necessary support can be considered dependent of, mainly, the geo-mechanic characteristics of the excavated medium. Only outside the range of intermediate tunnel depths is that other factors significantly impact the geo-mechanic behavior of the excavation: for deep excavations, the in-situ natural stresses of the medium in which the excavation is performed, and for shallow excavations, the cinematic-rigid equilibrium caused by the section's proximity to the external topographic surface.

1 INTRODUCTION

As it is widely known, there exist numerous and complex factors which for a specific tunnel section interact to determine its overall geostatic behavior. One of these factors is the in-situ stresses prior to excavation which, as a first approximation, can be related to the section's cover depth. Nevertheless, for a given tunnel it is possible to define a range of cover depths for which this factor's influence is irrelevant and therefore negligible for practical purposes.

This does not represent a new idea. In fact, several methodologies, traditional and more modern ones, are based on the acceptance of this concept. Some times to determine the loads acting on the tunnel support, and in other instances to define the support to be selected: Bierbaumer (1913), Terzaghi (1946), Protodyakonov (1960), Wickham (1972) and Bieniawsky (1973), are some of the most significant examples on the application of the theory and practice in tunneling

Although following different procedures and criteria, all these methods estimate the tunnel loads and support type based on the geomechanic characteristics of the ground in which the excavation is taking place, and on the dimensions of the tunnel section.

In spite of their ease of use and therefore, the profound diffusion in practice, these methodologies have often revealed evident and irrefutable limitations. These limitations put in evidence the fact that, although the geomechanic characteristics of the ground are of paramount importance in the tunnel design, they alone are not able to capture all the necessary parameters to satisfactorily define the geostatic tunnel design.

In other words, there often exist circumstances in which, although the geotechnical characteristic of the excavated material are essentially the same, the geostatic tunnel conditions are effectively and significantly different. This fact demonstrates the

existence of other factors of influence. Among these factors, there certainly exist some important ones: in some cases, the pre-existing tensorial natural conditions of the ground to be excavated, or, in other cases, the near-by surface boundary conditions (geometric-topographic).

As a result, every tunnel should be identified based on two limiting cover depths: a lower and an upper bound. These boundaries can be significantly apart, which often implies that most sections of the tunnel fall inside the middle range. For this middle range, it is possible to assume that the geomechanic behavior of the section, the section loads and as a consequence the required support, can be designed based only on the geotechnical characteristics of the excavated material. As a consequence, the selection and definition of particular conditions becomes simpler, and so does the support design for each specific section, which can be easily selected and characterized.

On the other hand, outside of these limiting cover depths, that is, outside the intermediate sections, other aspects can significantly influence the geomechanic behavior of the excavation: for sections with large cover depth (deep sections), the deformations and stresses related to the pre-existing tensorial natural conditions of the ground to be excavated, and for sections with low cover depth (shallow sections), the rigid-kinematics equilibrium related to the proximity of the section to the surface.

Finally, it is important to anticipate that it is not possible to determine absolute values, which can be generally and universally selected for the two limiting depths. In fact, these values may differ from tunnel to tunnel, because they are a function of the shape and dimensions of the excavation, as well as, the geotechnical characteristics of the material to be excavated: the better the geomechanical characteristics of the medium, then the larger the upper limiting cover depth may become. That is, the more competent the geomaterials are, the larger the range of intermediate depths in which the support design is essentially controlled by, or dominated by, the geotechnical characteristic of the material alone.

2 BEHAVIOR CATEGORY

The geostatic behavior of an underground excavation, or more schematically, the type of behavior of the excavation, depends on a number of factors. On an extreme simplification, these factors can be identified as the in-situ natural conditions of the medium prior to the excavation, as well as its geomechanic resistance.

The natural in-situ stress state, as a first approximation, and when additional measurements are not available, can be related to the excavation depth or cover (H). The geomechanic characteristics of the medium can also, in an approximate manner, be related to the resistance of the dominant materials in the ground, as well as to the geomechanical macro structure or the rock mass (fractures, weathering, anisotropy and discontinuities morphology, among others).

To identify and define such geomechanic characteristics of the medium several geomechanical quality indexes can be initially used. These include the RMR (Bieniawsky, 1973), the Q value (Barton, 1974), the RSR (Wikham, 1972), etc., or the more recent GSI [Geological Strength Index] (Hoek, 1994) and RMi (Palmstrom, 1995).

In a simplified approach, for those situations in which the in-situ conditions result in considerably high stresses with respect to the natural rock mass resistance, it is possible to refer directly to the unconfined compressive strength of the rock mass (σ_{cm}) and compare it to the natural in-situ stress (γH), where (γ) is the rock mass density. These two quantities are related by an important parameter: the competence index of the excavation ($IC = \sigma_{cm} / \gamma H$). This index can become very helpful in defining the excavation behavior under the previously described conditions. On the other hand, in those situations in which the competence index (IC) is high, which is usually the case under moderate cover depths where the in-situ stresses are generally low, the quality of the rock mass (e.g. GSI) alone can become the discriminatory parameter to determine the type of behavior for the excavation.

BEHAVIOR CATEGORY and PRE-SELECTED SUPPORT TYPE

CATE GORY	BEHAVIOR	COVER DEPTH		PRESELECTED SUPPORT TYPE <i>(Approx tunnel diameter of 10m)</i>
		HIGH IC	LOW GSI	
A	Front Stability Cavity Stability Isolated Instabilities (block kinematics) $FS_f > 2.5$ $FSc > 2.5$ $\varepsilon < 1\%$ $\varepsilon_o \ll 0.5\%$ (Rp/Ro = 1)	> 0.45	> 60	Shortcrete (5-10 cm) + Bolts L = 4 m (if necessary)
B	Front Stability Cavity Slightly Instable $FS_f \approx 2$ $FSc \approx 1$ $1\% < \varepsilon < 2.5\%$ $\varepsilon_o \leq 0.5\%$ (1 < Rp/Ro < 2)	0.3 - 0.45	40 - 60	Shortcrete (10-15 cm) + Bolts (L = 4 - 6 m) (density 0.25/m ²) or, Light Ribs @ 1.5 m
C	Front Close to Equilibrium Cavity Instability $FS_f \approx 1$ $FSc < 1$ $2.5\% < \varepsilon < 5\%$ $0.5\% < \varepsilon_o < 1\%$ (2 < Rp/Ro < 4)	0.2 - 0.3	30 - 50	Shortcrete (15-20 cm) + Bolts (L = 6 m) (density 0.5/m ²) or, Medium Ribs @ 1m + Front Reinforcement (if necessary)
D	Front Instability Cavity Instability (large deformations) $FS_f < 1$ $FSc \ll 1$ $5\% < \varepsilon < 10\%$ $\varepsilon_o > 1\%$ (Rp/Ro > 4)	0.15 - 0.2	20 - 40	Shortcrete (20-25 cm) + Heavy Ribs @ 1m or, Bolts (L=6-9 m) (density 1/m ²) + Face Reinforcement + Additional Bolts (if necessary)
E	General Instability (very weak rock mass and/or fault zones) $FS_f \ll 1$ $FSc \ll 1$ $\varepsilon > 10\%$ $\varepsilon_o \gg 1\%$ (Rp/Ro >> 4)	< 0.15	< 20	Shortcrete (20-30 cm) + Very Heavy Ribs @ 1m + Face Reinforcement + Additional Bolts + Pre-support (if necessary)

Following this framework, and for practical purposes, the type of possible excavation behavior can, in principle, be separated into at least five categories. They can be identified, for example, by the uppercase letters A to E. These categories define a specific excavation behavior, which are ranked based on increasing quality characteristics. The quality characteristic can be selected as a function of a group of several parameters which can be either estimated and observed, or even measured: factors of safety of the excavation front and cavity ($FS_f - FS_c$), the front and cavity convergence ($\epsilon_o - \epsilon$), the plastic radius (R_p), the GSI and the IC, among others (G. Russo et Al., 1998).

The following table summarizes the most important characteristics, peculiarities and properties of each one of these five excavation behavior classes. It also includes a pre-selection of the support type associated to each category.

3 DESIGN LOADS

The qualitative pre-selection of the necessary support to warranty the required stability of the tunnel are based on the previously described excavation behavior categories. Following this pre-selection, it is necessary to proceed with the detailed analysis and structural design. This step is based on the estimation of the design loads acting on the support structure, as well as the calibration of the stiffness of the rock mass in which the structure will be placed.

As previously described and emphasized, to determine the loads acting on the support of a particular tunnel section it is convenient and necessary to group the tunnel sections into categories according to the cover depth range (H): low – intermediate – high.

The previously identified lower limit of the cover section (H_i) represents that value of the cover depth which delimits the shallow sections from the intermediate-depth sections. For the same tunnel, the previously identified upper limit of the cover section (H_s) represents that value of the cover depth which delimits the deep sections from the intermediate-depth sections.

Extensive experience has been gained in the design and construction of many kilometers of conventionally excavated tunnels. The geologic environment in which these tunnels have been constructed vary from those characterized by foliated metamorphic rock masses with significant heterogeneous physical conditions (from fresh to decomposed) to those masses of large rock blocks with equally variably physical conditions, and passing through residual and sedimentary rocks. This extensive experience has allowed the identification of the limiting cover depths in the range of 75 to 150m for H_s , and between 10 and 25m for H_i .

For each particular case, the specific value of the limiting depths depends on the section dimensions (e.g. width or equivalent diameter “b”) and the geomechanic characteristics of the soil mass (e.g. geomechanic group “GGi”, where “i” is the group from 1 to 5, which selection can be initially based on the Hoek geomechanic quality index “GSI”).

As a first approximation, it is possible to estimate, with reasonable accuracy, the values of the two limiting depths according to the following expressions:

$$H_i = b (50/GSI) \quad H_s = b (GSI/5)$$

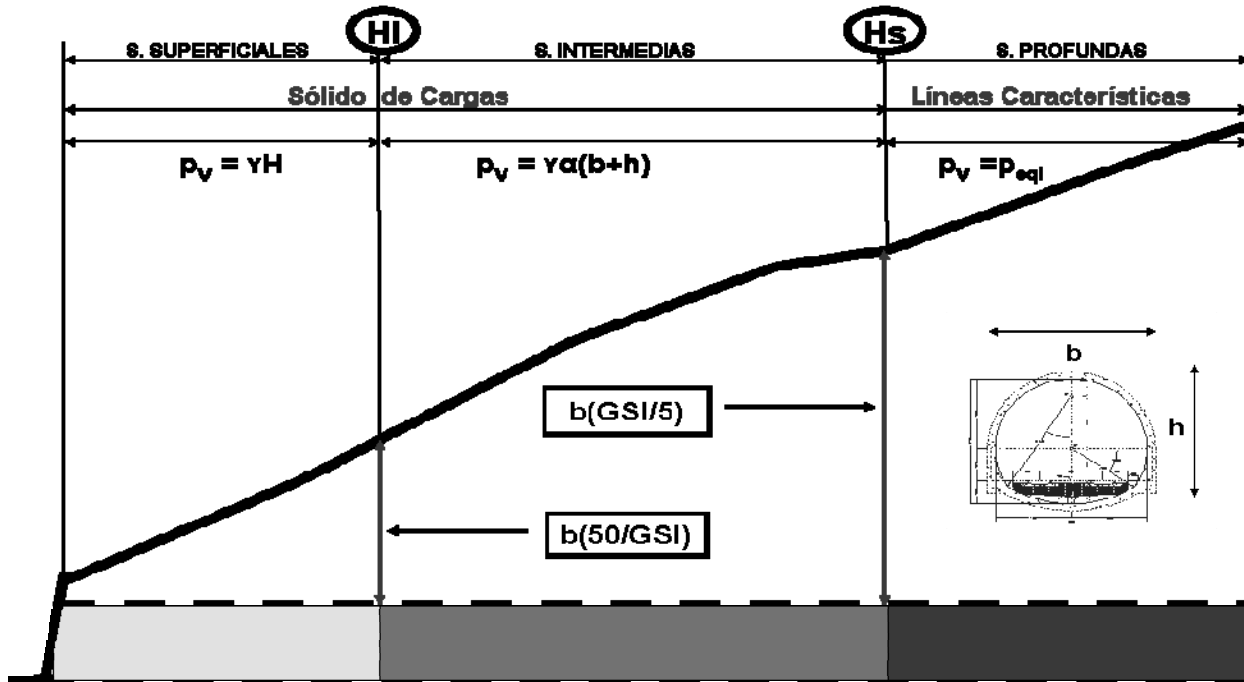
It can be noticed that, as previously explained, as the geomechanic quality of the rock mass increases, so does the range of intermediate cover depths (H_i decreases, while H_s increases). For these intermediate depths, the section’s geomechanic behavior, the behavior category for the excavation, and finally, the support to be implemented, can be associated, for a given dimension of the section, to the geomechanic characteristics of the rock mass: the GGi, or as a first approximation, the GSI directly.

In practice, according to the cover depth of a given tunnel section, the loads acting on the support can be generally estimated following one of two different methodologies: “ground arch loads” method for cover depths shallower than H_s , and “characteristics lines” method in the deeper cases in which the cover depth is greater than H_s .

Furthermore, a different load distribution scheme is applied: for shallow sections, the final support sustains the vertical gravity loads on the crown, and horizontal loads in the walls; for deeper sections, the loads act in the radial direction only on the crown. For the primary support, the applied load is generally modeled by the simplified radial loads on the crown and walls.

height is equal to: $H_p = \alpha(b+h)$, where ‘ α ’ is a proportionality coefficient (Terzaghi), which is a function of the geomechanic characteristics of the ground, ‘ b ’ is the section width and ‘ h ’ the section height.

The coefficient (α) is a function of the ‘GSI’ and ‘ m_i ’ (the mechanical index for the intact rock defined in the Hoek & Brown failure criterion).



- For those sections of low cover depth, classified as “shallow sections” ($H \leq H_i$), the equilibrium load acting on the primary support, and the vertical loads acting on the final support are the same, and will be equal to the gravity forces (γH), corresponding to a solid which height is equal to the specific cover depth.

The design horizontal load on the final support of these sections will be equal to those forces resulting from the classical theory of soil loading on retaining structures. Seismic loads will also be included.

- For those sections of moderate cover depth, and classified as “intermediate sections” ($H_i < H \leq H_s$), the equilibrium load acting on the primary support will be equal to the gravity forces of a solid which

Its value can be approximately estimated by (Perri, 2002):

$$\alpha = 1244 m_i^{-1.433} GSI^{(0.0004 m_i m_i - 0.0046 m_i - 1.2344)}$$

In order to estimate the vertical loads acting on the final support of those sections classified as “intermediate,” it is possible to eventually reduce the value of the ‘ α ’ coefficient by about 25% (and maximum 50%) of the value estimated from the above expression. The reduction magnitude depends on the geomechanic conditions of the excavation and the expected time length before the placement of the final support.

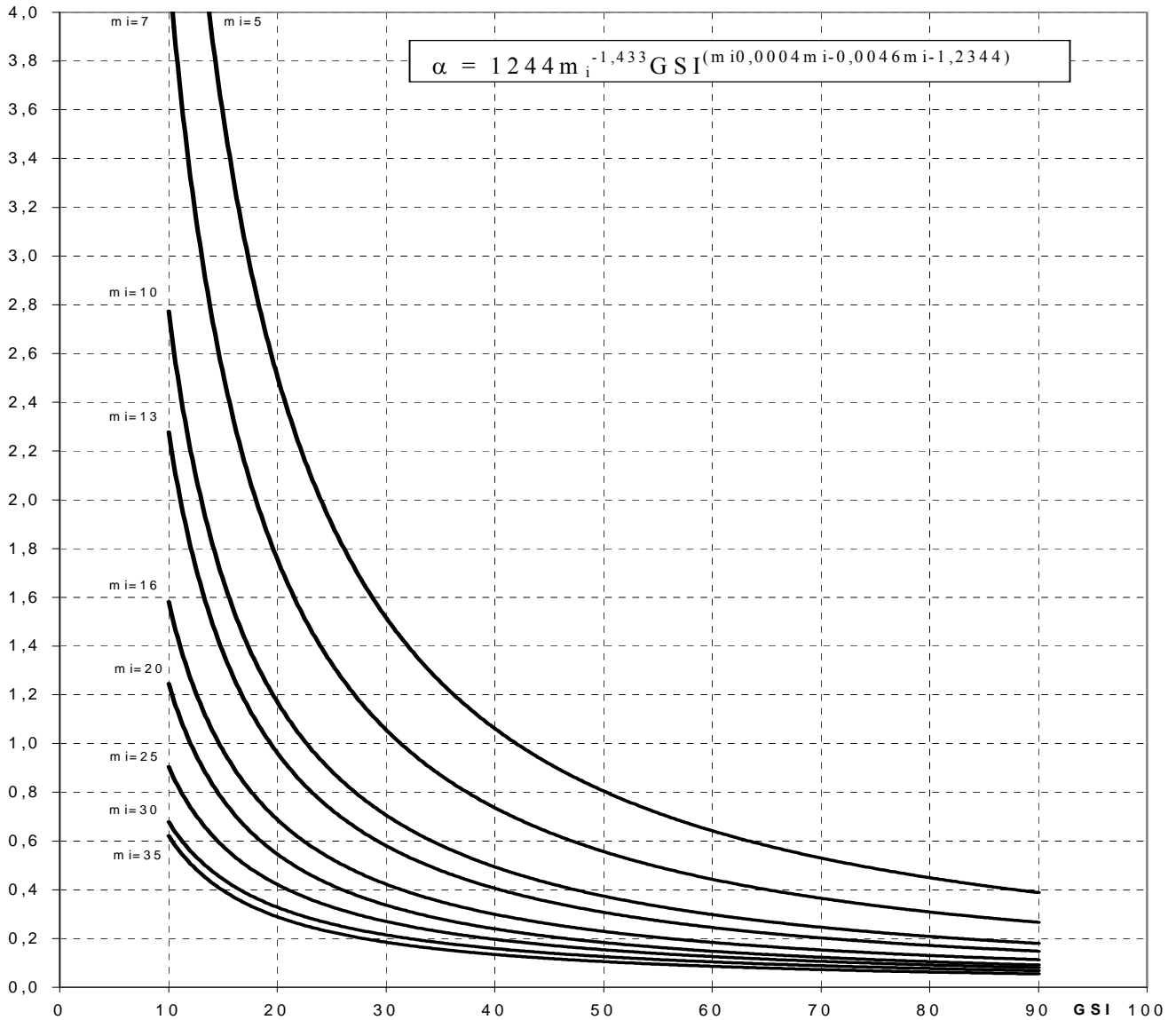
In fact, the magnitude of the reduction coefficient will be larger if it is possible to assume, with a certain degree of confidence, that the primary

support is in fact loaded by the ground arch loads before the final support construction. In this case, the final support will only be loaded by that portion of the load that is not being resisted by the primary support.

Seismic loading will be applied in those circumstances in which the geologic and geotechnical studies explicitly recommend them.

FACTOR DE CARGAS "α" DE TERZAGHI

(Perri, 2002)



Depending on the selected model for the analysis, the horizontal load acting on the final support of these intermediate sections will be equal to those forces resulting from the classical theory of soil loading on retaining structures or those derived from the elastic confining resistance of the ground acting on the deformable support.

- For those sections that can be classified as "deep sections" (H > H_s), the equilibrium loads acting on the primary support are obtained from the characteristic lines interaction analysis.

The design loads on the final support will be applied in the radial direction, only in the crown, and their magnitude will be proportional to the plastic radius extension, as calculated in the equilibrium analysis, or the radius extension that is expected before the primary support effectively starts to work.

The horizontal loads will be those derived from the elastic confining resistance of the ground acting on the deformable support.

Seismic loading will be applied in those circumstances in which the geologic and geotechnical studies explicitly recommend them.

For the calculation of the loads acting on the structural support, all the previously described steps must also be considered in the final structural design. This final structural design must be based on the specific capacities of the available supports for each project.

PRIMARY SUPPORT

Modern tunnel technology has evolved towards a system of elements that comprise the excavation support. The main element is the fiber-reinforced shotcrete, complemented if necessary, by metallic ribs and bolts. The metallic ribs and bolts may be placed, depending on the situation, in different combinations.

An example combination is shown at the end of this paper, in which five typical primary supports are described (SP-A; SP-B; SP-C; SP-D; SP-E) for an approximately 10 meter wide tunnel.

From the table it can be noticed that, with the exception of the two extreme situations (one in which, because of the geomechanic conditions it is not necessary to include metallic ribs in the support, and the other in which, on the other extreme, it is practically impossible to avoid their use), in all other intermediate situations, which comprise the most recurrent situations in tunneling practice, it is always possible to choose among two alternative support technologies: the first one, based on the use of metallic ribs to integrate the shotcrete and the second one, based

on the systematic and extensive use of metallic bolts also used to integrate the shotcrete.

In fact, structurally speaking, it is possible to achieve the same structural or contrast support capacity using either one of the alternatives. As a consequence, the practical selection of either alternative will depend on factors such as: site availability of the different support elements, site availability of the required installation equipment, cost of each alternative, contractual conditions, productivity and contractor experience among other.

It is possible to elaborate a list of technical advantages and disadvantages of either alternative, but this could easily become a strongly subjective discussion.

Finally, based on the alternative support considered or available for each project, the support to be used must be selected for each design section. In this manner it is now possible to compare the expected loads (demand), according to the cover depth and possible geomechanical conditions of the rock mass to be encountered, with the capacities of the available support alternatives.

Applying the previously described procedure, the following table summarizes two results for an approximately 10 meter wide (or equivalent diameter) tunnel. This table shows the support selection as a function of the excavation behavior, which is related to the GSI for shallow cover depths ($H \leq H_s$), and to the Competence Index (IC) for deeper sections ($H > H_s$):

$$IC = \sigma_{cm} / \gamma H = \frac{0.0034 \cdot m_i^{0.8} \cdot \sigma_{ci} \cdot [1.029 + 0.025 \cdot \exp(-0.1 \cdot m_i)]^{GSI}}{\gamma H}$$

EXAMPLES OF PRIMARY SUPPORT FOR 10 m WIDE TUNNELS

<i>COVER DEPTH $\leq H_i$</i>		<i>H_i < COVER DEPTH $\leq H_s$</i>		<i>COVER DEPTH > H_s</i>	
GSI ≤ 20	SP-D	GSI ≤ 20	SP-E	IC ≤ 0.15	SP-E
20 < GSI ≤ 40	SP-C	20 < GSI ≤ 40	SP-D	0.15 < IC ≤ 0.20	SP-D
30 < GSI ≤ 50	SP-C	30 < GSI ≤ 50	SP-C	0.20 < IC ≤ 0.30	SP-C
40 < GSI ≤ 60	SP-C	40 < GSI ≤ 60	SP-B	0.30 < IC ≤ 0.45	SP-B
GSI > 50	SP-B	GSI > 50	SP-A	IC > 0.45	SP-A

<i>Diámetro Túnel b=10m</i>	H $\leq H_i$			H_i < H $\leq H_s$	H > H_s			
	H ≤ 10	10-20	20-30	30-40	40-60	60-80	80-100	H > 100
GSI ≤ 20	SP-E	SP-E	SP-E	SP-E	IC			
20 < GSI ≤ 30	SP-E	SP-E	SP-D	SP-D				
30 < GSI ≤ 50	SP-D	SP-D	SP-C	SP-C				
50 < GSI ≤ 60	SP-C	SP-B	SP-B	SP-B				
GSI > 60	SP-C	SP-A	SP-A	SP-A				

<i>Support Type</i>	<i>Shotcrete</i>	<i>Metallic Ribs</i>	<i>Metallic Bolts 20 tons</i>	<i>Capacity (MPa)</i>
SP-A	10 cm	-	-	0.15
SP-B	14 cm	2 IPN140 @ 150 cm	2 x 4 m @ ribs pair	0.25
		o, alternatively 7 bolts x 4m @ 150 cm (without ribs)		
SP-C	16 cm	2 IPN160 @ 125 cm	4 x 6 m @ ribs pair	0.35
		o, alternatively 11 bolts x 6m @ 125 cm (without ribs)		
SP-D	20 cm	2 IPN200 @ 100 cm	6 x 6 m @ ribs pair	0.45
		o, alternatively 15 bolts x 6m @ 100 cm (without ribs)		
SP-E	20 cm	2 IPN200 @ 075 cm	11 x 6 m @ ribs pair	0.55

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