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Estimation of rock mass deformation modulus and strength of jointed hard rock masses using the GSI system

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Abstract

Rock mass characterization is required for many applications in rock engineering practice including excavation design, support design, stope design, amongst others. For these purposes, it is necessary to obtain design input parameters such as deformation moduli and strength parameters for numerical modeling. Although such parameters can ultimately be determined from in situ tests, at the preliminary design stage, where access to underground is limited, the practical way to obtain these parameters is to apply a rock mass classification system to characterize the rock mass and estimate the rock mass properties. Over the years, many classification systems, such as RQD, Rock Mass Rating, Q and Geological Strength Index (GSI) systems, have been developed. Amongst them, the Q system is widely used for rock support system design and the GSI system is used for estimating design parameters. The GSI system is the only rock mass classification system that is directly linked to engineering parameters such as Mohr–Coulomb, Hoek–Brown strength parameters or rock mass modulus. However, the application of the existing GSI system is hindered by the facts that the use of the system is to some extent subjective and requires long-term experience.

In the present study, a quantitative approach to assist in the use of the GSI system is presented. It employs the block volume and a joint condition factor as quantitative characterization factors. The approach is built on the linkage between descriptive geological terms and measurable field parameters such as joint spacing and joint roughness. The newly developed approach adds quantitative means to facilitate use of the system, especially by inexperienced engineers.

The GSI system is applied to characterize the jointed rock masses at two underground powerhouse cavern sites in Japan. GSI values are obtained from block volume and joint condition factor, which in turn are determined from site construction documents and field mapping data. Based on GSI values and intact rock strength properties, equivalent Mohr–Coulomb strength parameters and elastic modulus of the jointed rock mass are calculated and compared to in situ test results. The point estimate method is implemented to approximate the mean and variance of the mechanical properties of the jointed rock masses. It is seen that both the means and variances of strength and deformation parameters predicted from the GSI system are in good agreement with field test data.

1. Introduction

In recent years, there has been renewed interest internationally in the construction of large-scale underground powerhouses and nuclear waste repositories as well as in the mining of massive orebodies at depth. To design, construct and mine these underground excavations safely and economically, it is important to know the rock mass properties thoroughly, and to further develop tools, methods and technologies leading to practically useful means for design.

The rock mass deformation modulus and strength are required as inputs to analyze the rock mass behavior by numerical models. The determination of the global mechanical properties of a jointed rock mass remains one of the most difficult tasks in the field of rock mechanics. Because there are so many parameters that affect the deformability and strength of an arbitrary rock mass, it is generally impossible to develop a universal law that can be used in any practical way to

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predict the strength of the rock mass. Traditional methods to determine these parameters in Japan and other countries include plate-loading tests for deformation modulus and in situ block shear tests for strength parameters. These tests can only be performed when the exploration audits are excavated and the cost of conducting in situ tests is high. Few attempts have been made to develop methods to characterize the jointed rock mass to estimate the deformability and strength indirectly. The Geological Strength Index (GSI), developed by Hoek et al. [1], is one of them. It uses properties of intact rock and jointing to determine/estimate the rock mass deformability and strength. GSI values can be estimated based on the geological description of the rock mass and this is well suited for rock mass characterization without direct access to the rock mass from tunnels. The GSI system concentrates on the description of two factors, structure and block surface conditions (Fig. 1). Although it has been used extensively in many countries, its applicability to the rock masses in Japan has not been tested, primarily because it seeks quantitative output from qualitative input and requires extensive engineering experiences and judgment. Although imperfect, the GSI system is the only system that provides a complete set of mechanical properties (Hoek–Brown strength parameters m_b and s, or the equivalent Mohr–Coulomb strength parameters c and ϕ, as well as elastic modulus E) for design purpose.

In the present study, upon the request from many engineers, efforts have been made to quantify GSI system parameters to better classify jointed rock masses for engineering purpose, and to develop a supplementary approach, which is quantitative in nature and easy to use. The approach is built on the linkage between descriptive geological terms and measurable field parameters such as joint spacing and joint roughness. The newly developed quantitative approach assists in gaining consistent ratings from parameters measurable during field mapping.

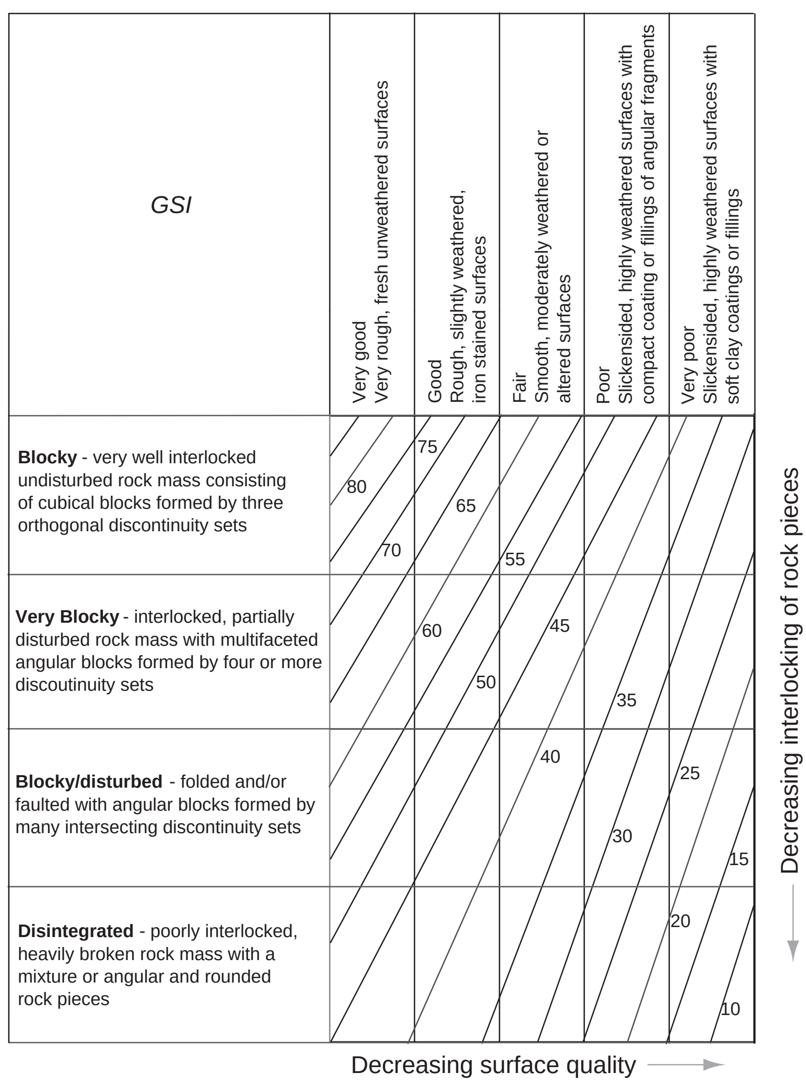


Fig. 1. Original GSI chart (reproduced from [18]).

The GSI system is then applied to characterize the jointed rock masses at two underground powerhouse cavern sites in Japan. GSI values are obtained from block volume and joint condition factor, which in turn are determined from site construction documents and field mapping data. Based on the resulting GSI values and intact rock strength properties, equivalent Mohr–Coulomb strength parameters and elastic modulus of the jointed rock mass are calculated and compared to in situ test results. The point estimate method (PEM) is implemented to approximate the mean and variance of the mechanical properties of the jointed rock masses.

2. Rock mass characterization for mechanical properties of rock masses

2.1. Summary of rock mass classification systems

Rock mass characterization is the process of collecting and analyzing qualitative and quantitative data that provide indices and descriptive terms of the geometrical and mechanical properties of a rock mass. Ideally, rock mass classification should provide a quick means to estimate the support requirement and to estimate the strength and deformation properties of the rock mass. A rock mass classification scheme is intended to classify the rock masses, provide a basis for estimating deformation and strength properties, supply quantitative data for support estimation, and present a platform for communication between exploration, design and construction groups.

A summary of the roles that rock mass classification systems play is presented in Fig. 2. The focus of this study is on the use of a rock mass classification system (GSI) to estimate the mechanical properties of jointed rock masses.

Many rock mass classification systems have been proposed and used in engineering practice, such as the RQD [2], Rock Mass Rating (RMR) [3], Q [4,5], GSI [1,6], and RMi system [7,8]. Some systems are based on the modification of the existing ones to suit specific application. For example, the RMR system was modified by Laubscher [9] for mine design and by Kendorski et al. [10] for drift support design in caving mines. The Q-system was modified by Potvin [11] for stope design. In Japan, the rock mass classification system developed at the Central Research Institute of Electric Powers (hereafter referred as "Denken system") for dam and underground cavern construction is widely used [12–14]. The Denken system is primarily a rock mass grouping system. As discussed above, a rock mass classification system can be used to estimate mechanical properties at a preliminary design stage and thus is well suited for the planned deep underground nuclear waste disposal program in Japan which is in the site selection phase. Of the many alternatives, the GSI system seems to be the best choice for design because it can provide a complete suite of input parameters for numerical analysis of panel stability.

As can be seen from Table 1, there are more than a dozen parameters that should be considered when describing a rock mass and using the results for certain design purposes. If the main purpose of the rock mass classification is to group the rock mass and provide rock mass strength and deformation property estimates, then, only the inherent parameters are needed. The inherent parameters are identified as the parameters of intact rock, joints, and faults. In a design process that employs numerical analysis, rock mass deformation modulus and strength are the only required input parameters. Other parameters such as excavation shape, size, and in situ stress are considered separately in the numerical model. Furthermore, if the jointed rock masses and faults are treated separately, the most important parameters

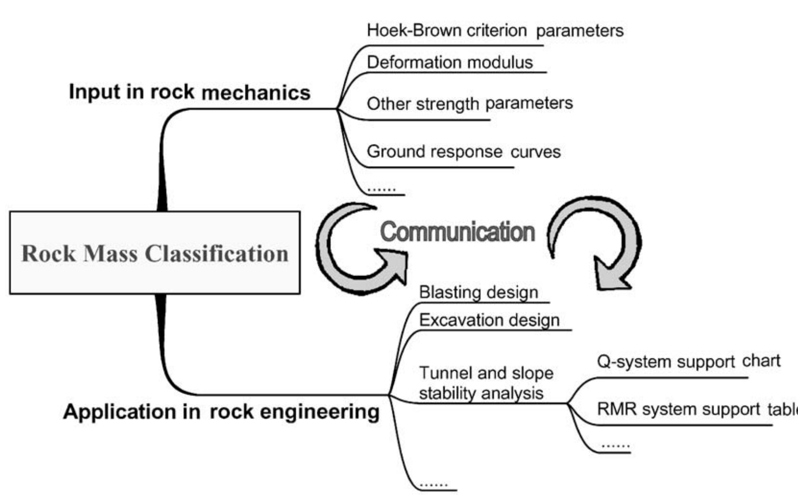


Fig. 2. Application of rock mass classification systems in rock mechanics and rock engineering.

Table 1
Important parameters for rock mass classification and characterization in engineering application

Group of parameters		Individual parameters
Rock mass inherent parameters	Intact rock parameters	*Strength of intact rock*
		Rock modulus
	Joint parameters	*Number of joint sets*
		Joint frequency
		Joint condition (roughness, infilling)
		Joint size/length, persistency
		Joint orientation
	Weak zones or faults	Width
		Orientation
		Gouge material (modulus and strength)
External parameters		In situ stress
		Ground water
Construction parameters		Excavation size
		Excavation shape
		Construction method
		Blasting damage

related to the determination of strength and deformation properties of jointed rock masses for input in a numerical analysis are highlighted in Table 1 (Italic). A characterization system that is designated for this purpose should consider only these parameters. It is observed that the GSI system fits the criterion imposed by the idea of a universal rock mass classification system [15]. It uses only a finite set of parameters or a Universal Parameter Set to characterize a rock mass, and with a rating in the range of 0–100.

2.2. Rock mass classification for mechanical properties of rock

2.2.1. Rock mass strength

Two types of strength criteria, i.e., Mohr–Coulomb and Hoek–Brown failure criteria, are widely used in rock engineering. The strength of a jointed rock mass depends on the strength of the intact rocks and the joint condition such as the shape of the intact rock pieces and the conditions of surfaces separating the blocks. In terms of major and minor principal stresses, σ_1 and σ_3, the Mohr–Coulomb failure criterion can be expressed as

$$\sigma_1 = \frac{2c\cos\phi}{1-\sin\phi} + \frac{1+\sin\phi}{1-\sin\phi}\sigma_3, \tag{1}$$

where c and ϕ are the cohesive strength and angle of friction of the rock mass, respectively. The generalized Hoek–Brown criterion for jointed rock masses [16] is

$$\sigma_1 = \sigma_3 + \sigma_c\left(m_b\frac{\sigma_3}{\sigma_c} + s\right)^a, \tag{2}$$

where m_b, s, a are constants for the rock mass, and σ_c is the uniaxial compressive strength of the intact rock. In order to apply the Hoek–Brown criterion for estimating the strength of jointed rock masses, three properties of the rock mass have to be estimated. The first one is the uniaxial compressive strength of the intact rock. The second is the value of the Hoek–Brown constant m_i for the intact rock and the last one is the value of GSI for the rock mass. Whenever possible, the values of σ_c and m_i should be determined by statistical analysis of the results of a set of triaxial tests on carefully prepared core samples. σ_c alone can be determined from uniaxial compressive tests. Simple index tests such as point load test and Schmidt hammer test, both can be carried out in field to estimate σ_c. When rock testing is not performed or the number of tests is limited, σ_c and m_i can be estimated from published tables. For details the reader is referred to [1,17,18].

The Mohr–Coulomb strength parameters, c and ϕ, can also be obtained from a series of block shear tests in exploration tunnels. Hoek–Brown strength parameters can be obtained in a similar fashion, but in situ triaxial tests are preferred. Beside the huge costs of such tests, it is often difficult to carry out large-scale triaxial tests to determine these parameters. In search for a practical solution, Hoek and Brown [19] recognized that the characteristics which control rock mass deformability and strength were similar to the characteristics adopted in Q and RMR rock mass classification systems and suggested that rock mass classification could be used to estimate the constants m_b and s. A table was proposed and widely accepted by the geotechnical community [19]. Experiences gained from using the table showed reasonable estimates on a large number of projects. In a later update, Hoek and Brown [16] suggested that the material parameters for a jointed rock mass (Eq. (2)) could be estimated from the modified 1976-version of Bieniawski's RMR [3], assuming completely dry conditions and a favorable joint orientation. Because this does not work for very poor rock with RMR less than 25, a new index called GSI was introduced [1]. In this manner, the GSI system consolidates various versions of the Hoek–Brown criterion into a single simplified and generalized criterion that covers all of the rock types normally encountered in underground engineering. A GSI value is determined from the structure interlocking and joint surface conditions (Fig. 1). It ranges from 0 to 100.

2.2.1.1. Rock yielding in a ductile manner. When GSI is known, the parameters in Eq. (2) are given as [20]

$$m_b = m_i \exp\left(\frac{\mathrm{GSI} - 100}{28 - 14D}\right), \tag{3}$$

$$s = \exp\left(\frac{\mathrm{GSI} - 100}{9 - 3D}\right), \tag{4}$$

$$a = 0.5 + \frac{1}{6}(\mathrm{e}^{-\mathrm{GSI}/15} - \mathrm{e}^{-20/3}), \tag{5}$$

where D is a factor that depends on the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. The D factor was introduced in the latest update [20] of the Hoek–Brown failure criterion. During the construction of hydropower caverns, careful excavation techniques with controlled blasting are applied ($D = 0$) and Eqs. (3) and (4) revert to those given in the early version of Hoek and Brown failure criterion [18].

The equivalent Mohr–Coulomb parameters can be obtained based on the Hoek–Brown envelope and a chosen range of σ_3-values (see Fig. 3). Hoek and Brown [18] suggested to use eight equally spaced values of σ_3 in the range of $0<\sigma_3<0.25\,\sigma_c$ to obtain c and ϕ. For hard rocks, e.g., $\sigma_c = 85$ MPa, this would imply a σ_3 range of 0–21 MPa. This range may not be suitable for underground excavation design where the confinement near the opening is small, usually in the range of 0 to $\leqslant 5$ MPa. A stress path from A to B, illustrated in Fig. 3, would indicate elastic response based on c and ϕ determined from $\sigma_3 = 0-5$ MPa range but would fail according to c and ϕ determined from $\sigma_3 = 0-21$ MPa range. It is therefore necessary to obtain the equivalent c and ϕ for this confinement range.

In the recent update, Hoek et al. [20] suggest to obtain the maximum confining levle ($\sigma_{3\max}$) for deep tunnels from the following equation:

$$\frac{\sigma_{3\max}}{\sigma_{\mathrm{cm}}} = 0.47\left(\frac{\sigma_{\mathrm{cm}}}{\gamma H}\right)^{-0.94}, \tag{6}$$

where σ_{cm} is the rock mass strength, γ is the unit weight of the rock mass and H is the overburden depth. A plot of Eq. (6) is shown in Fig. 4. It is seen that for caverns around 400 m deep, the $\sigma_{3\max}$ is around 5 MPa. A set

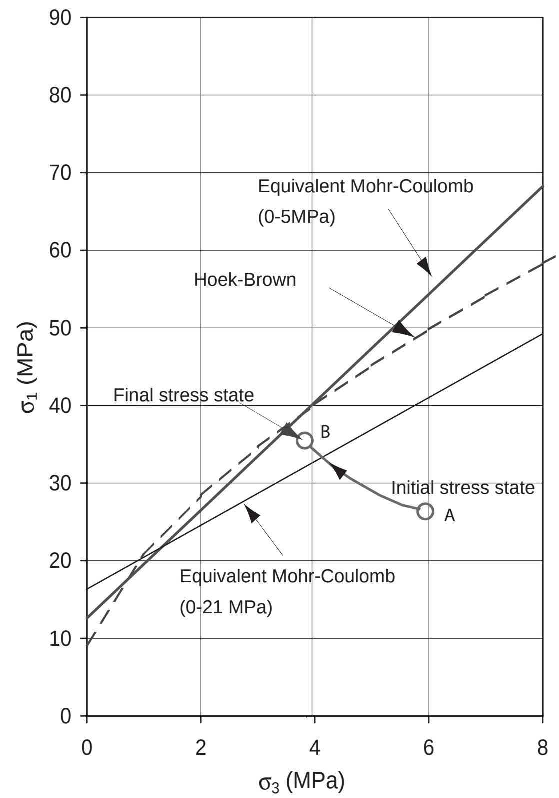


Fig. 3. Approximation of Mohr–Coulomb envelope from the Hoek–Brown envelope.

of curves for the equivalent c and ϕ within the $\sigma_{3\max} = 5$ MPa range is presented in Fig. 5. The resulting ϕ is higher and c is lower for a range $\sigma_3 = 0-5$ MPa as compared to a wider σ_3 range ($0<\sigma_3<0.25\,\sigma_c$) [17]. This lower confinement range is also in accordance with the normal stress usually applied to shear blocks during in situ shear block tests in Japan.

2.2.1.2. Rock failing in a brittle manner. Pelli et al. [21] found that the parameters obtained from Eqs. (3) and (4) did not predict the observed failure locations and extend near a tunnel in a cemented sand or siltstone. They found that lower m_b and higher s values were required to match predictions with observations. Further analyses of underground excavations in brittle rocks eventually lead to the development of brittle Hoek–Brown parameters ($m_b = 0$, $s = 0.11$) by Martin et al. [22] for massive to moderately fractured rock masses with tight interlocks that fail by spalling or slabbing rather than by shear failure. Accordingly, Eqs. (3) and (4) are clearly not applicable for GSI > 75 in massive to moderately or discontinuously jointed hard rocks. The zone of anticipated brittle failure conditions is highlighted in Fig. 7 by the hatch near the upper left corner. While further work is required to fine-tune the boundary between brittle failure by spalling and shear failure near excavations, empirical

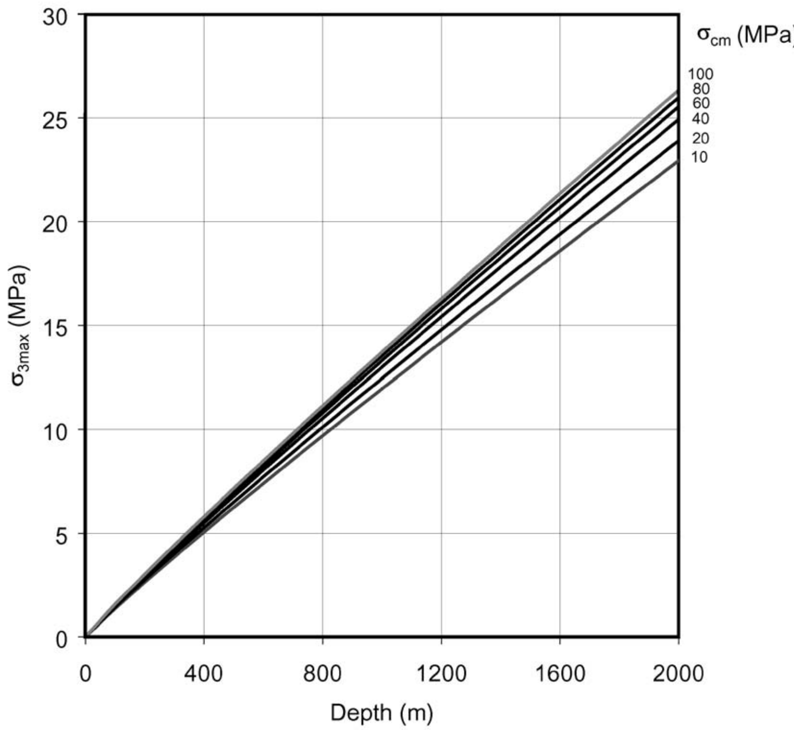


Fig. 4. Determination of σ_{3max} for equivalent Mohr–Coulomb parameter calculation for deep tunnels.

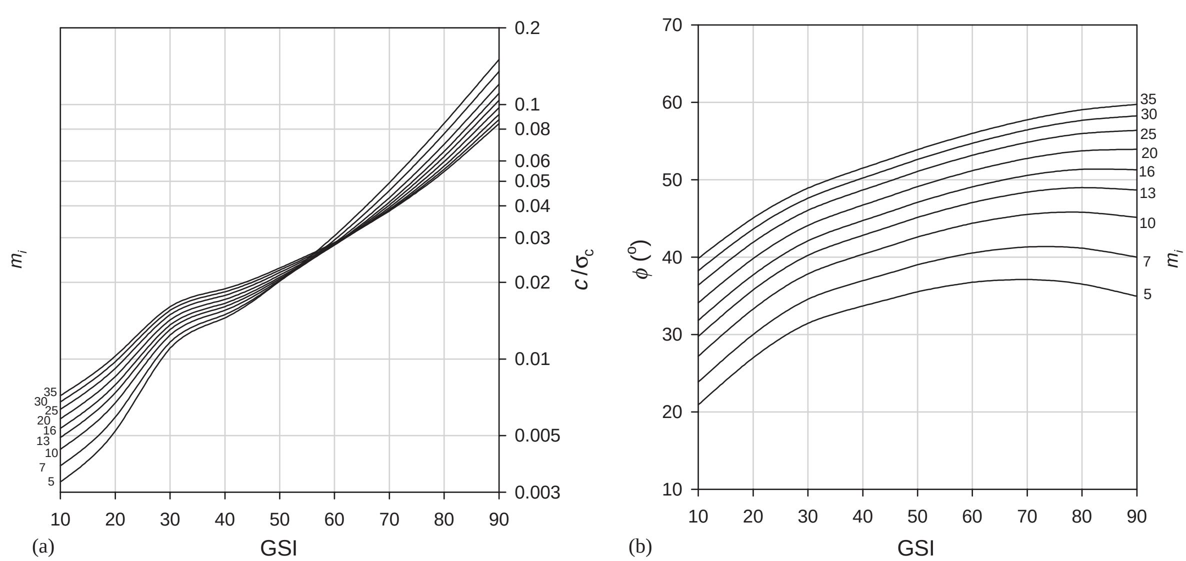


Fig. 5. Plots of: (a) cohesive strength; and (b) friction angles for different GSI and m_i values. Note that the confinement range assumed in obtaining these figures are from 0 to 5 MPa.

evidence suggests that brittle Hoek–Brown parameters [22] are applicable for strong rocks ($\sigma_c > 50$ MPa) with moderate to high modulus ratios ($E/\sigma_c > 200$; [2]) and $V_b > 10$–$100 \times 10^3\,\text{cm}^3$, $J_C > 1$–2 and GSI > 65–75, where V_b and J_C are block volume and joint condition factor, respectively. The definition of V_b and J_C as well as the details of the development of Fig. 7 are presented in Section 3.

2.2.2. Deformation

The mean deformation modulus is related to the GSI system [20] as

$$E = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_c}{100}}10^{((\mathrm{GSI}-10)/40)}\ \mathrm{GPa}$$

for $\sigma_c < 100$ MPa. (7)

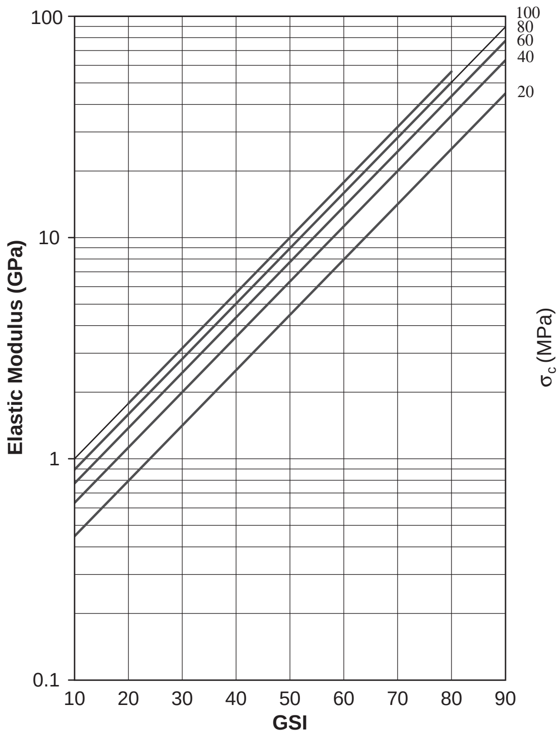


Fig. 6. Plots of elastic modulus for different GSI and σ_c values.

The variations of E as a function of GSI and σ_c are plotted in Fig. 6 for $D = 0$. The inclusion of σ_c in Eq. (7) shows the influence of the modulus of the intact rock (E_0) on the deformation modulus of the rock mass. Good correlation between the modulus E_0 and σ_c of the intact rock exists [2].

3. Determination of GSI based on block volume and joint surface condition factor

3.1. Quantification of the GSI system

The GSI system has been developed and evolved over many years based on practical experience and field observations. GSI is estimated based on geological descriptions of the rock mass involving two factors, rock structure or block size and joint or block surface conditions. Although careful consideration has been given to the precise wording for each category and to the relative weights assigned to each combination of structural and surface conditions, the use of the GSI table/chart (Fig. 1) involves some subjectivity. Hence, long-term experiences and sound judgment is required to successfully apply the GSI system.

To overcome these difficulties, a different approach building on the concept of block size and conditions, namely, the idea of block volume and joint condition factor is proposed in the present study. The resulting approach adds quantitative measures in an attempt to render the system more objective. By adding measurable quantitative input for quantitative output, the system becomes less dependent on experience while maintaining its overall simplicity.

The proposed GSI chart is presented in Fig. 7. The descriptive block size is supplemented with the quantitative block volume (V_b) and the descriptive joint condition is supplemented with the quantitative joint condition factor (J_C). The influence of V_b and J_C on GSI was calibrated using published data and then applied to two caverns for verification based on back-analysis. Note that the original GSI chart covers only four structure categories, i.e., blocky, very blocky, blocky/disturbed, disintegrated. Extensions to include a "massive" category for large block volumes and moderately jointed rock, and "foliated/laminated/sheared" category for very small block volumes or highly fractured rock are included in Fig. 7. These extensions of rock block categories agree with the recent update of the GSI chart [23].

3.1.1. Block volume

Block size, which is determined from the joint spacing, joint orientation, number of joint sets and joint persistence, is an extremely important indicator of rock mass quality. Block size is a volumetric expression of joint density. In the cases that three or more joint sets are present and joints are persistent, the volume size can be calculated as

$$V_b = \frac{s_1 s_2 s_3}{\sin\gamma_1 \sin\gamma_2 \sin\gamma_3}, \tag{8}$$

where s_i and γ_i are the joint spacing and the angle between joint sets, respectively (Fig. 8). Random joints may affect the shape and size of the block. Statistically, joint spacing follows a negative exponential distribution. For a rhombohedral block, the block volume is usually larger than that of cubic blocks with the same joint spacings. However, compared to the variation in joint spacing, the effect of the intersection angle between joint sets is relatively small. Hence, for practical purpose, the block volume can be approximated as

$$V_b = s_1\, s_2\, s_3. \tag{9}$$

When irregular jointing is encountered, it is difficult to delineate three or more joint sets. In these cases, the block volume can be directly measured in the field by selecting some representative blocks and measuring their relevant dimensions. Other methods using RQD, the volumetric joint count J_V [24], and weighted joint density can also be used. An example is given in Section 4.1.

If the joints are not persistent, i.e., with rock bridges, the rock mass strength is higher and the global rock stability is enhanced. Consequently, the apparent block volume should be larger for rock masses with

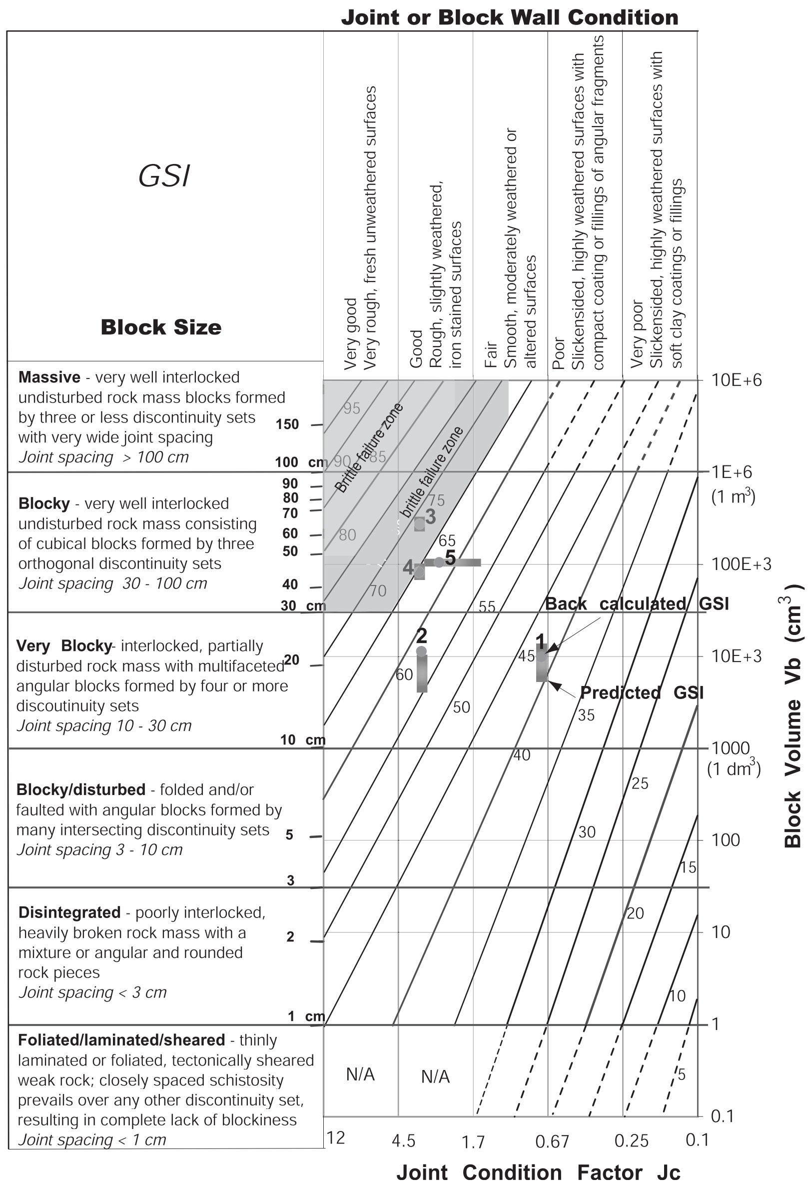


Fig. 7. Quantification of GSI chart.

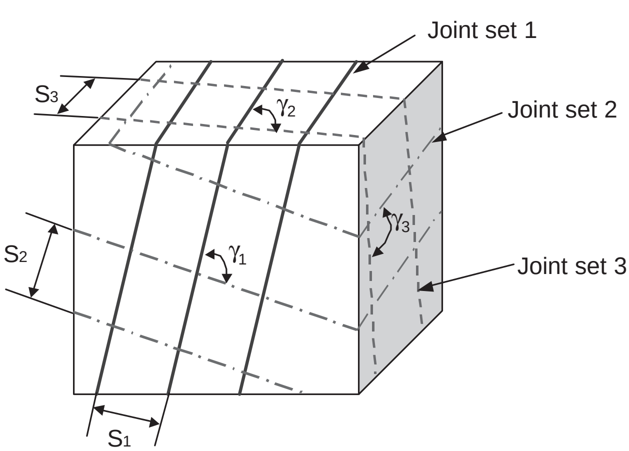


Fig. 8. Block delimited by three joint sets.

non-persistent joints. The presence of discontinuous joints has a significant effect on the properties and behavior of rock masses and should be included in the engineering characterization. Diederichs and Kaiser [25] demonstrated that the capacity of 1% rock bridge area equivalent to 10×10 cm or 100 cm^2 rock bridge per 1 m^2 total area in a strong rock (UCS > 200 MPa) is equivalent to the capacity of at least one cablebolt. The joint persistence is considered in the GSI system by the block interlocking description. In the following, a joint persistence factor is proposed to quantify the degree of interlocking. It should be kept in mind that in reality, the determination of joint persistence is deemed difficult, either from mapping in underground drift or surface outcrops.

If s_i and $\bar{l}_i$ are the average joint spacing and the accumulated joint length of set i in the sampling plane, respectively, and L is the characteristic length of the rock mass under consideration, a joint persistence factor p_i is defined as

$$p_i = \begin{cases} \dfrac{\bar{l}_i}{L} & \bar{l}_i < L, \\ 1 & \bar{l}_i \geqslant L. \end{cases} \tag{10}$$

Because the joints are discontinuous, an equivalent spacing for continuous joint has to be found to use Eq. (8) to calculate the block volume. Based on the consideration that short joints are insignificant to the stability of the underground excavation with a larger span, or are insignificant to the rock mass properties with a longer characteristic length [26], the equivalent spacing for discontinuous joints is defined as

$$s_i' = \frac{s_i}{\sqrt[3]{p_i}} \tag{11}$$

and the equivalent block volume is expressed as

$$V_b = \frac{s_1 s_2 s_3}{\sqrt[3]{p_1 p_2 p_3}\,\sin\gamma_1 \sin\gamma_2 \sin\gamma_3}. \tag{12}$$

Examples: Suppose that there are three orthogonal joints and the characteristic length is 10 m. If the average joint length is 2 m, then $V_b/V_b^0 = 5$ (V_b^0 is the block volume by assuming persistent joint sets), which means that the equivalent block volume with discontinuous joints is 5 times larger than that with persistent joints. Eq. (12) is proposed to consider the joint persistence in this study but further work is required to collect field data to fine-tune the equations.

3.1.2. Joint condition factor

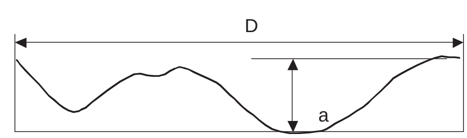
In the GSI system, the joint surface condition is defined by the roughness, weathering and infilling condition. The combination of these factors defines the strength of a joint or block surface. In this study, we propose to use a joint condition factor, similar to the factor used by Palmstrøm [27], to quantify the joint surface condition. This joint condition factor, J_C, is defined as

$$J_C = \frac{J_W J_S}{J_A}, \tag{13}$$

where J_W and J_S are the large-scale waviness (in meters from 1 to 10 m) [27] and small-scale smoothness (in centimeters from 1 to 20 cm) [27] and J_A is the joint alteration factor. The ratings from the Q and RMi-system are adopted here and are listed in Tables 2, 3 and 4 for J_W, J_S and J_A, respectively. The waviness (Table 2) is measured by the undulation expressed as a percentage. According to Barton and Bandis [28], both the large-scale (waviness) and small-scale (uneveness) roughness can be estimated by the amplitude of the asperities (a). In reality, because the small-scale asperities have a base length of some centimeter and the

Table 2
Terms to describe large-scale waviness [27]

Waviness terms	Undulation	Rating for waviness J_W
Interlocking (large-scale)		3
Stepped		2.5
Large undulation	$>3\%$	2
Small to moderate undulation	0.3–3%	1.5
Planar	$<0.3\%$	1



Undulation = a/D
D - length between maximum amplitudes

Table 3
Terms to describe small-scale smoothness [27]

Smoothness terms	Description	Rating for smoothness J_S
Very rough	Near vertical steps and ridges occur with interlocking effect on the joint surface	3
Rough	Some ridge and side-angle are evident; asperities are clearly visible; discontinuity surface feels very abrasive (rougher than sandpaper grade 30)	2
Slightly rough	Asperities on the discontinuity surfaces are distinguishable and can be felt (like sandpaper grade 30–300)	1.5
Smooth	Surface appear smooth and feels so to touch (smoother than sandpaper grade 300)	1
Polished	Visual evidence of polishing exists. This is often seen in coating of chlorite and specially talc	0.75
Slickensided	Polished and striated surface that results from sliding along a fault surface or other movement surface	0.6–1.5

Table 4
Rating for the joint alteration factor J_A [4,27]

	Term	Description	J_A
Rock wall contact	*Clear joints*		
	Healed or "welded" joints (unweathered)	Softening, impermeable filling (quartz, epidote, etc.)	0.75
	Fresh rock walls (unweathered)	No coating or filling on joint surface, except for staining	1
	Alteration of joint wall: slightly to moderately weathered	The joint surface exhibits one class higher alteration than the rock	2
	Alteration of joint wall: highly weathered	The joint surface exhibits two classes higher alteration than the rock	4
	Coating or thin filling		
	Sand, silt, calcite, etc.	Coating of frictional material without clay	3
	Clay, chlorite, talc, etc.	Coating of softening and cohesive minerals	4
Filled joints with partial or no contact between the rock wall surfaces	Sand, silt, calcite, etc.	Filling of frictional material without clay	4
	Compacted clay materials	"Hard" filling of softening and cohesive materials	6
	Soft clay materials	Medium to low over-consolidation of filling	8
	Swelling clay materials	Filling material exhibits swelling properties	8–12

Table 5
Case studies

	V_b ($\times 10^3\,cm^3$)	Joint condition factor J_C	GSI	
			Predicted	Back-calculated
Case 1 Granite	5–15	Rough, small undulating, hard chlorite coating $J_S = 2, J_W = 1.5, J_A = 4, J_C = 0.75$	40–45	42
Case 2 Palaeozoic siltstone	5–10	Slightly rough, undulating, fresh without coating or infilling $J_S = 1.5, J_W = 2, J_A = 1, J_C = 3$	55–58	58
Case 3 Andesite at Kidd Creek Mine	351 (average)	Rough, undulating, fresh with some quartz infillings $J_S = 2, J_W = 2, J_A = 1, J_C = 4$	68	69
Case 4 Basalt at Holt–McDermott Mine	80 (average)	Rough, undulating, fresh $J_S = 2, J_W = 2, J_A = 1, J_C = 4$	64	65
Case 5 Gjovik Olympic hall, Norway	100 (average)	Smooth to rough, undulating; infilling of clay, chlorite, silt/sand, calcite in a few joints $J_S = 1.5, J_W = 2, J_A = 1-2, J_C = 1.5-3$	57–65	64

amplitudes are on the order of hundreds of millimeters that are difficult to measure, a descriptive rating system is provided in Table 3. The joint alteration factor (Table 4) alone has the most impact on the joint condition factor as it can reduce it by more than one order of magnitude.

3.2. Examples

The values of GSI predicted from the GSI chart and the ones back-calculated from other systems are presented in Table 5 and Fig. 7. Cases 1 and 2 stem from the thesis of Palmstrøm [27], Cases 3 and 4 from underground mapping of two mine sites in Canada (unpublished data at GRC), and Case 5 is from the well-known Gjovik Olympic Hall, Norway [29].

It is seen from the above examples that the proposed addition of block volume and joint condition factor to the GSI chart fit the existing data well. The quantitative system provides a supplementary representation of the qualitative structure and joint condition descriptions, assisting less experienced engineers in arriving at consistent ratings. The block volume spectrum from "massive" to "very blocky" rock masses ranges from $1\,m^3$ to $1\,dm^3$ and for "disturbed" to "sheared" rock from 1000 to $<1\,cm^3$. The joint condition factor J_C ranges from 0.1 to 12. With the information of joint spacing, roughness, and alteration, one can easily pin down a specific structure category. In this fashion, the accuracy to determine GSI value for rock masses in different jointing range has been greatly improved.

The addition of block volume V_b and joint condition factor J_C to the GSI chart represents the quantification of the original qualitative system (Fig. 1). It is not a substitute for the descriptive approach but rather intended as a supplement to ensure consistent application. There are situations that may render the quantified approach difficult to be applied; For example, in rock masses that are disintegrated, foliated, or sheared. For these types of rocks, the descriptive approach still provides the only means for strength and deformation parameter estimation [23]. Also, at the feasibility investigation stage where quantitative data are not available, the descriptive approach is still applicable. However, as the site investigation produces data from core and borehole geophysical logging, as well as from field mapping, the quantitative system takes over to simplify the characterization process for consistent results. In the next section, the quantitative GSI chart (Fig. 7) is used to estimate the mechanical properties at two cavern sites in Japan. As it will be seen later, the quantitative approach allows us to consider the variability of the strength and deformation parameters. The estimated values, both mean and standard deviation, are then compared to the in situ test data to demonstrate the applicability of the system to jointed rock masses.

4. Application

4.1. Rock mass characterization at two cavern sites in Japan

4.1.1. Kannagawa site

Kannagawa pumped hydropower project [30] in Gumma Prefecture in Japan is now under construction with a maximum output of 2700 MW. The powerhouse cavern at 500 m depth has a width of 33 m, a height of 52 m, and a length of 216 m. The cavern excavation was started in 1998 and the last bench was completed in 2000.

The rock mass at the site consists of conglomerate, sandstone, and mudstone. The rock masses are classified into five major groups as shown in Fig. 9. The percentage of conglomerate in CG1 and CG2 rock mass zones are about 93% and 62%, respectively. To use the GSI system to characterize the rock masses, lab tests results on intact rocks are used along with field mapping data. One of the long-standing challenges in analyzing rock strength and deformation data is the fact that these values are quite variable. Since the intact rock strength, joint spacing, and joint surface condition vary even within the same rock type designation zone, the PEM [31] is used to represent the encountered variability of rock mass properties. PEM is an alternative to Monte Carlo simulation with models containing a limited number of uncertain inputs. In this method, the model is evaluated at a discrete set of points in the uncertain parameter space, with the mean and variance of model predictions computed using a weighted average of these functional evaluations. This is important because rock mass properties such as strength and modulus vary from site to site and from point to point and average values alone do not represent encountered conditions well.

Sixty-four uniaxial compressive tests were conducted and the average and standard deviation of each rock type are presented in Table 6 (only data for four rock types, i.e., CG1, CG2, FS1, M1, are shown). The parameter m_i for each rock types was obtained from a

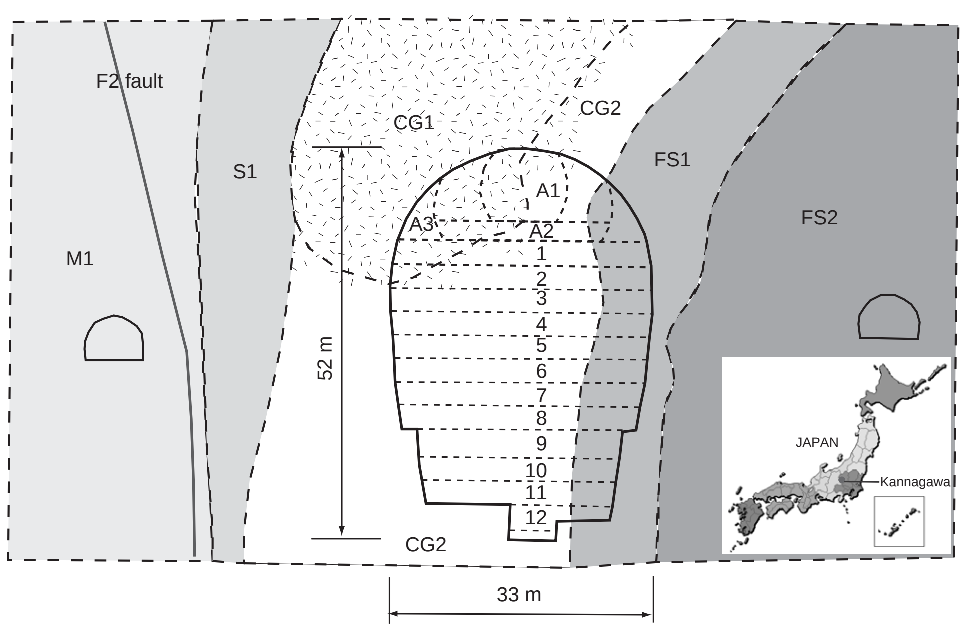


Fig. 9. Geological cross-section at Kannagawa project showing different zones of rock masses. The inset shows the geographical location of the project site in Japan.

Table 6
Characterization of the rock masses at the Kannagawa site using the GSI system

	CG1			CG2			FS1			M1		
	GSI system		Test data[a]	GSI system		Test data[a]	GSI system		Test data[a]	GSI system		Test data[a]
	Ave.	Std. dev.		Ave.	Std. dev.		Ave.	Std. dev.		Ave.	Std. dev.	
UCS (MPa)	111	15.3		162	34.3		126	24.9		48	4.8	
m_i	22	2.75		19	2.38		19	2.38		9	1.125	
Joint freq. (#/m)	0.74	—		0.84	—		1.03	—		3.8	—	
RQD	99.7			99.6			99.4			93.4		
β	31	1.33		31	1.33		31	1.33		31	1.33	
V_b ($\times 10^5$ cm^3)	3.09	0.13		3.03	0.13		2.95	0.13		1.10	0.05	
J_W	2.5	0.2		1.5	0.12		1.5	0.12		1.5	0.12	
J_S	2	0.16		1.5	0.12		1.5	0.12		1.5	0.12	
J_A	1	0.08		1	0.08		1	0.08		2	0.08	
J_C	5.03	0.70		2.26	0.31		2.26	0.31		1.13	0.16	
Estimated GSI	74	1.8		65	1.3		65	1.3		54	1.7	
c (MPa)	4.11	0.58	5.2	3.67	0.63	3.4	3.05	0.47	3.4	1.14	0.09	1.9
ϕ (deg.)	57.8	1.37	57	57.5	1.54	57	56.4	1.55	57	41.9	1.49	40
E (Gpa)	39.6	4.11	45.3 (6.2)[b]	23.5	1.76	33 (3.6)[b]	23.5	1.76	24.4 (2.5)[b]	8.7	0.96	11.7 (1.7)[b]

[a] Rock mass strength test data c and ϕ are from in situ block shear tests and elastic modulus test data E is from plate-load tests.
[b] Standard deviation from plate loading test data.

limited number of triaxial tests. The coefficient of variation (Cov) of m_i was chosen as 15% according to Hoek [17]. Joint frequencies in zones CG1 and CG2 are 0.74 and 0.85 joint/m, respectively. The average joint frequency is 1.1 joint/m in FS1 zone, and 3.7 joint/m in M1 zone. Joint density is low at this site, which makes it difficult to delineate distinct joint sets. This is a common problem when characterizing massive rock masses. There are two ways to overcome this problem. One is to directly observe the rock block size on site and the other is to estimate block size indirectly using joint frequency from scan-line mappings. The second method is used here to account for the sparse joint distribution. The block size is estimated from [27] $V_b = \beta((115 - \mathrm{RQD})/(3.3))^{-3}$, where RQD is calculated from the joint frequency.[1] β is the block shape factor ranging from 27 to over 100. For equal-dimensional blocks, the average is $\beta = 31$ with a 27–35 range. The assumption of equal-dimensional block shape is based on in situ observations made by the authors during a site visit.

Furthermore, during the site visit to Kannagawa powerhouse construction site, the joint conditions were rated. In CG1 zone, the joints are stepped in large-scale and rough in small-scale with no weathering; in CG2, FS1/FS2 and M1 zones, the joints are moderately undulated with slightly rough surfaces and have no alteration; joints in M1 zone are moderately altered. Naturally, joints with different roughness and alteration can be observed in the various rock mass zones. To account for the variation in geology and uncertainty involved in field observation, it is assumed that the coefficients of variation for joint roughness and alteration are 8%. This is based on observations of the ratings for J_S and J_A that showed possible errors in assessing joint roughness and alteration of ± 0.25 or more.

Because little information is available about the joint frequency variation, the average and standard deviation of V_b is calculated considering the variation of β only. Based on the PEM and using the GSI chart, the average and standard deviation of GSI is obtained using two variables V_b and J_C. The resulting coefficients of variation of GSI are in the range of 2–3.2%. The averages and standard deviations of the equivalent Mohr–Coulomb parameters and the elastic modulus are calculated based on σ_c, m_i, and GSI. All the results for different rock mass zones are presented in Table 6. Also shown in Table 6 are the strength parameters c and ϕ determined from 21 in situ block shear tests and the deformation modulus E determined from 29 in situ plate-load tests.

The density distribution function for c and ϕ are plotted in Fig. 10. Because no distribution data can be obtained from one set of in situ block shear test, only the average c and ϕ values are plotted in these figures. These two figures should be used simultaneously as the two parameters c and ϕ are not independent variables. Fig. 11 presents the strength envelopes for zones CG2, FS1, and M1. In general, the average envelopes from the GSI system are very close to these obtained from in situ shear tests. The shaded areas of the strength envelopes are obtained assuming combinations of $c+$ and $\phi+$, $c-$ and $\phi-$, respectively. Here, $c+$ and $c-$ mean the values of average c plus or minus one standard deviation of c, respectively. It is seen that the GSI system slightly underestimates c for M1. Fig. 12 presents a comparison

[1] $\mathrm{RQD} = 100\,e^{-\lambda t}(1 + \lambda t)$, where λ is the fracture frequency and t is a threshold level. In most cases, $t = 0.1$ m is used.

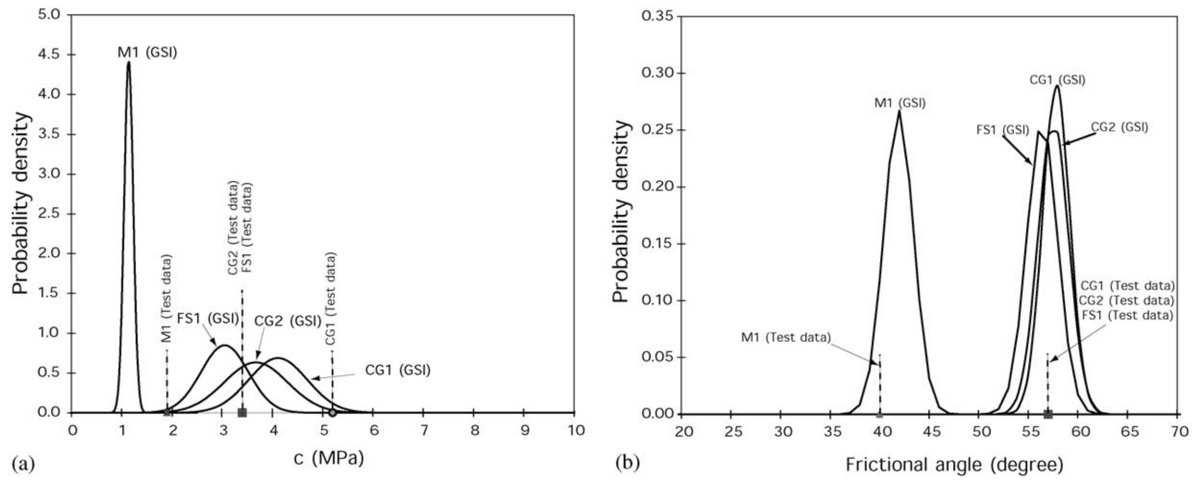


Fig. 10. Comparison of: (a) cohesion; and (b) friction angle distributions calculated from the GSI system and field test data at the Kannagawa site.

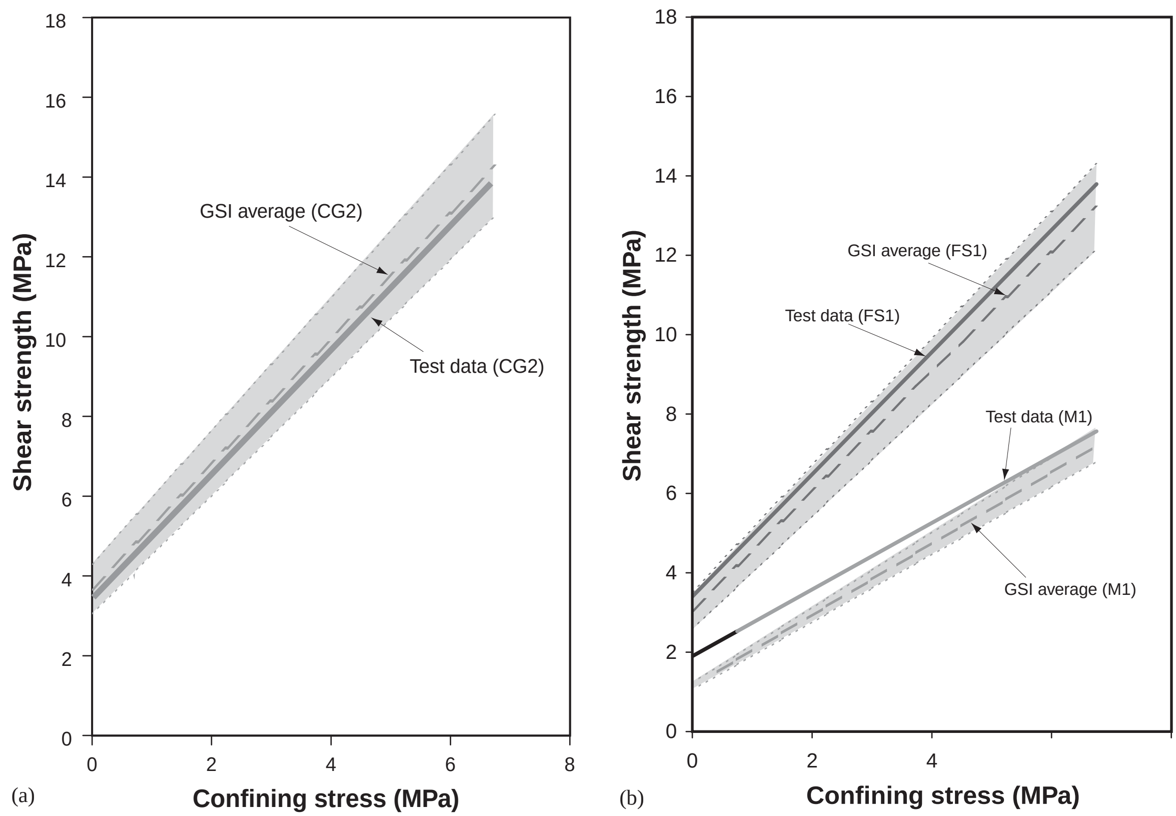


Fig. 11. (a, b) Comparison of shear strengths of zones CG2, FS1 and M1 from the GSI system and field test data at the Kannagawa site.

of elastic moduli obtained from the GSI system and from in situ plate-load test. The GSI system leads to underestimate the rock modulus when compared to plate-load tests. However, it is reasonable to assume that the rock mass modulus is lower than that of a local plate load test. The predicted ranges are generally smaller than those obtained from plate-loading test.

4.1.2. Kazunogawa site

Kazunogawa power station [32], located in Yamanashi Prefecture, Japan (Fig. 13), at about 500 m depth, has a generating capacity of 1600 MW. The cavern dimensions are: width 34 m, height 54 m, and length 210 m. The cavern excavation was started in 1994 and the last bench was excavated in 1996.

The rock mass consists of sandstone and composite rock of sandstone and mudstone, described as two groups (C_H and C_M) of rock mass types based on the Denken system [12]. 75 uniaxial compressive tests were conducted and the average and standard deviation of each rock type are presented in Table 7. Three joint sets are observed at this site. The joint spacing of the major joint set is in the range of 1–20 cm. The average joint spacings of the other two joint sets are 25 and 50 cm, respectively. Joints are fresh, have small undulation and are rough. The joint surface assessment is supported by joint profiles obtained by using laser scanner in laboratory tests. The rock grouping (C_H and C_M) and the block sizes are basically controlled by the joint frequency of the major joint set. From joint density distribution graphs, the average joint spacing is about 10 cm for C_H and about 2.5 cm for C_M.

Joint spacing usually follows a negative exponential distribution [33]. To account for the uncertainty involved in the geological information, it is assumed that the coefficient of variation for log joint spacing is about 10%, and the coefficients of variation for joint large-scale waviness, small-scale smoothness, and joint alteration are 8%.

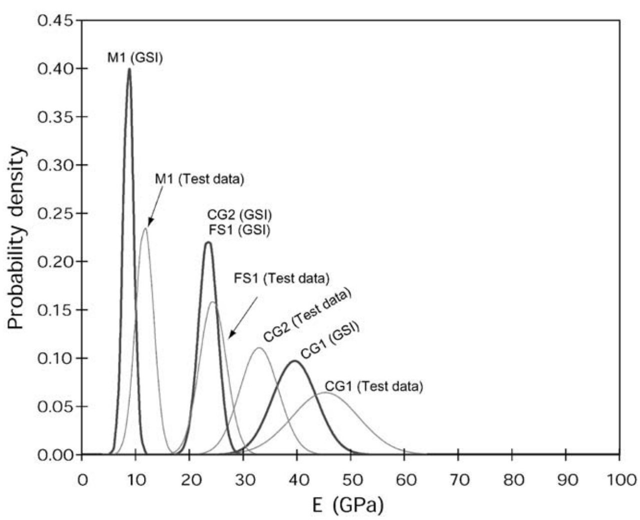


Fig. 12. Comparison of elastic modulus distributions calculated from the GSI system and field test data at the Kannagawa site.

The PEM is used again to consider the variability taking the joint spacing distribution of each joint set into account. Based on the PEM and using the GSI chart, the average and standard deviation of GSI are obtained. The coefficients of variation of GSI for C_H and C_M rock masses are 4.1% and 3.5%, respectively. The averages and standard deviations of the equivalent Mohr–Coulomb parameters and the elastic modulus are calculated based on σ_c, m_i, and GSI. The results for the two rock types are presented in Table 7 along with c and ϕ determined from 12 in situ block shear tests and deformation moduli determined from 29 in situ plate-load tests.

The density distribution functions for c and ϕ are plotted in Fig. 14. Because no field distribution data is available, only the average c and ϕ in situ values are plotted in these figures. Fig. 15 presents the strength envelopes for C_H and C_M rock masses. In general, the average envelopes from the GSI system are very close to these obtained from in situ shear test. The shaded areas of the strength envelopes are plotted using the same method explained earlier. Fig. 16 presents a comparison of elastic moduli obtained from the GSI system to those from in situ plate-loading test. The GSI system over-estimates the modulus for C_H rock mass by about 29% and underestimates the modulus for C_M rock mass by about 8%. Despite of this, the GSI system predicts the

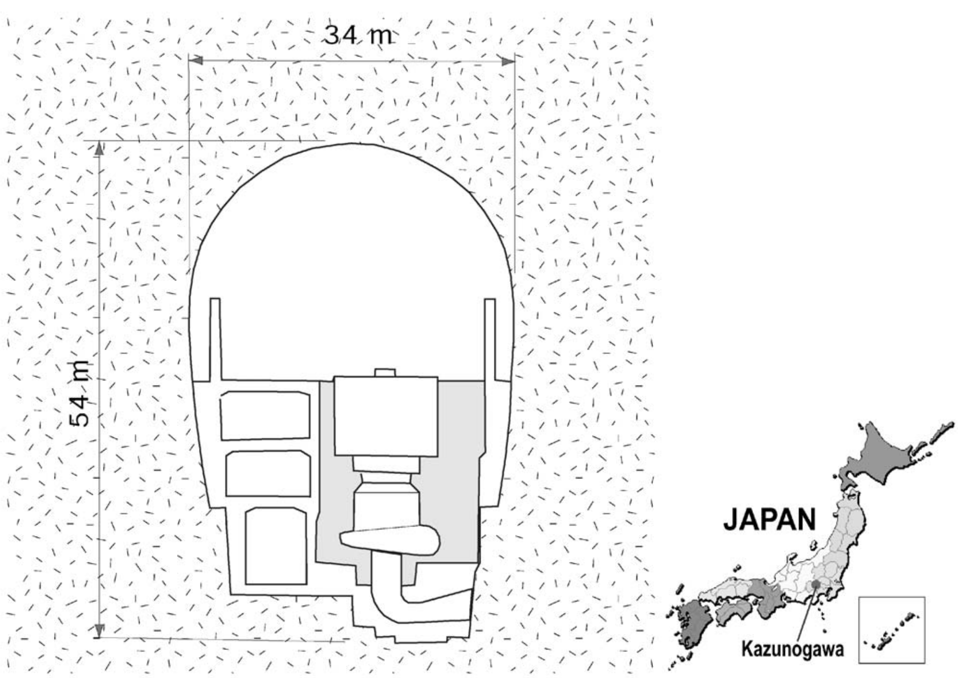


Fig. 13. Cross-section of the Kazunogawa powerhouse cavern. The inset shows the geographical location of the project site in Japan.

Table 7
Characterization of the rock masses at the Kazunogawa site using the GSI system

	C_H				C_M			
	GSI system			Test data[a]	GSI system			Test data[a]
	Ave.	Std. dev.	Log (Std. dev.)		Ave.	Std. dev.	Log (Std. dev.)	
UCS (MPa)	108	42			108	42		
m_I	19	2.375			19	2.375		
s_1 (cm)	10		0.10		2.5		0.04	
s_2 (cm)	25		0.14		25		0.14	
s_3 (cm)	50		0.17		50		0.17	
V_b (cm^3)	1.25×104		0.242		3.125×103		0.224	
J_W	2	0.167			1.5	0.167		
J_S	2	0.167			2	0.167		
J_A	1	0.08			2	0		
J_C	4.0	0.56			1.5	0.21		
Estimated GSI	60	2.5			46	1.6		
c (MPa)	2.29	0.64		1.5	1.41	0.30		0.8
ϕ (°)	54.7	2.57		58	52.5	2.94		55
E (GPa)	16.7	2.93		12.9 (2.84)[b]	7.3	1.01		7.9 (1.22)[b]

[a] Rock mass strength test data c and ϕ are from in situ block shear tests and elastic modulus test data E is from plate-load tests.
[b] Standard deviation from plate-loading test data.

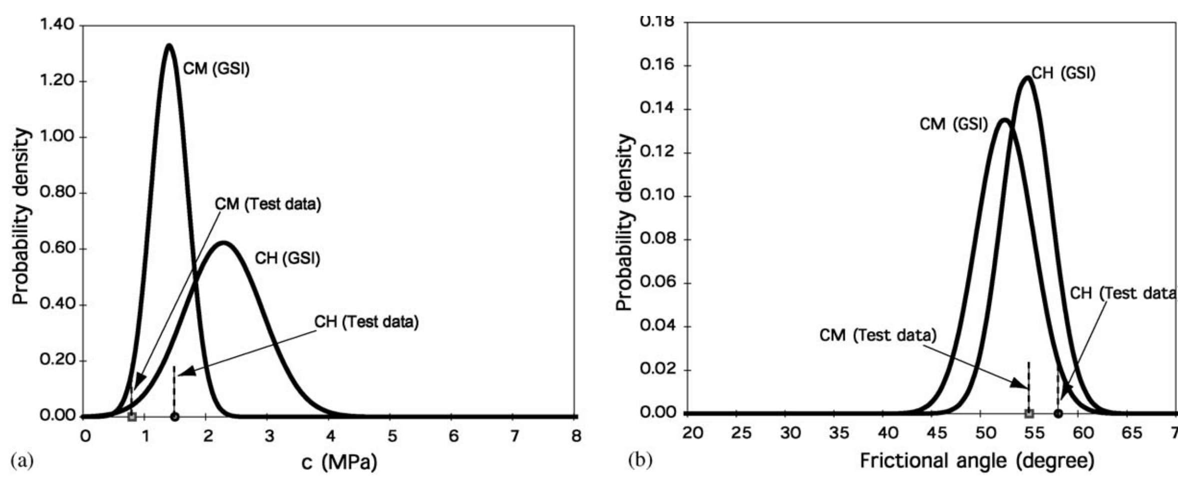


Fig. 14. Comparison of (a) cohesion; and (b) frictional angle distributions calculated from the GSI system and field test data at the Kazunogawa site.

elastic moduli distribution well when compared to the distribution of plate-loading test data.

4.2. Discussion of results

Traditionally, the determination of mechanical properties of jointed rock masses in Japan is achieved through well planned and executed in situ block shear test and plate-loading test. Such tests are expensive and time consuming. Most importantly, results only become available once underground access has been established. An alternative to the test approach is the use of a rock mass classification system such as the GSI system to provide design parameters early in the design phase and reduce the need for extensive in situ testing. Nevertheless, in situ tests can be used to verify the GSI prediction or the observational method [34] will be required to confirm the GSI predictions.

The proposed quantitative approach uses the block volume and joint surface condition factor to determine the GSI value. These input parameters were obtained from field mapping and from borehole logging data. The strength and deformation parameters estimated from the GSI system are very close to those obtained from in situ tests, indicating that the GSI system can be effectively applied to the design of underground caverns. One advantage of the quantitative approach is that the variability of inherent parameters can be explicitly considered in the calculation process. The variability of c, ϕ, and E can be implemented in the design tools to calculate the variability of stress and deformations as well as anticipated loads in rockbolts and anchors.

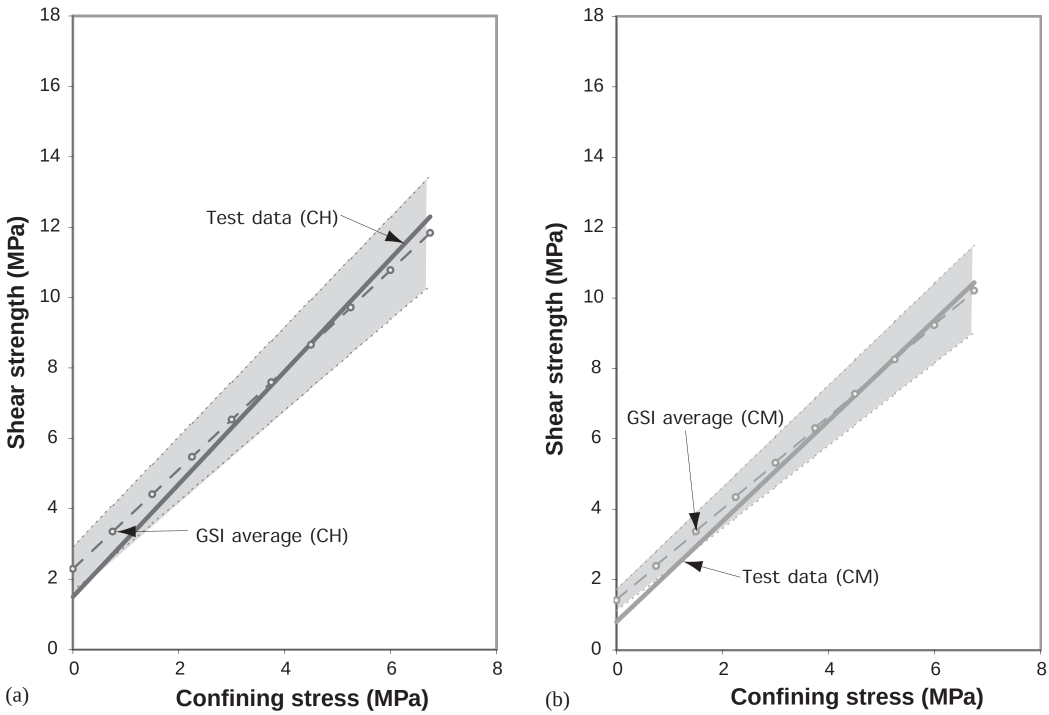


Fig. 15. (a, b) Comparison of shear strengths of rock masses C_H and C_M from the GSI system and field test data at Kazunogawa site.

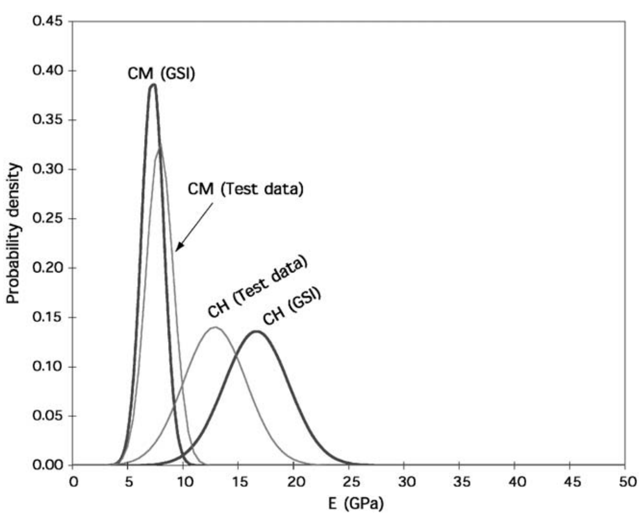


Fig. 16. Comparison of elastic modulus distributions calculated from the GSI system and field test data at the Kazunogawa site.

5. Conclusions

The GSI system is a universal rock mass classification system. It is the only rock mass classification system that is directly linked to engineering parameters such as Mohr–Coulomb or Hoek–Brown strength parameters or rock mass modulus. The GSI system can be used at all stages of a project but it is especially useful at the preliminary design stage where only limited information is available.

The original GSI system is based on a descriptive approach, rendering the system somewhat subjective and difficult to use for inexperienced personnel. To assist the use of the GSI system, a supplementary quantified approach for the GSI system is proposed in the present study by incorporating quantitative measures of block volume and joint condition factor. The structure description is supplemented by the block volume and joint condition description is supplemented by joint condition factor. Both axes were calibrated using published and newly collected data. The block volume can be calculated, in most cases, from joint spacings of three dominant joint sets. The joint condition factor is obtained by rating joint roughness depending on the large-scale waviness, small-scale smoothness of joints, and joint alteration depending on the weathering and infillings in joints.

The GSI system was applied to characterize the jointed rock masses at Kannagawa and Kazunogawa underground powerhouses in Japan. Based on the estimated GSI values and intact rock strength properties, equivalent Mohr–Coulomb strength parameters and elastic modulus of the jointed rock mass were calculated and compared to in situ test results. The PEM was applied to approximate variance of the mechanical properties of the jointed rock masses. It is found that both the means and variances of c, ϕ, and E predicted from the quantified GSI approach are generally in good agreement with field data. Hence, the quantitative approach added to the GSI system provides a means for consistent rock mass characterization and thus improves the utility of the GSI system.

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