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Correlation of the Hoek–Brown failure criterion for a sparsely jointed rock mass with an extended plane of weakness theory

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ABSTRACT

An appropriate estimate of rock mass strength is necessary for the design of civil and mining structures built in or on rock. Rock mass is an inhomogeneous and anisotropic material with complex behaviour, which contains random planes of discontinuities that tend to reduce its strength. The direct estimation of this strength is practically unfeasible, due to difficulties in sampling and testing. This has led to the development of empirical failure criteria. These, express the strength of the rock mass in terms of properties of the intact rock and the discontinuities. The Hoek–Brown criterion is the most widely accepted one. However, albeit its use for many years, no experimental in situ validation with the actual rock mass strength has been demonstrated. Therefore, the Hoek–Brown criterion is investigated analytically through an extended plane of weakness theory, already validated with experimental evidence on physical specimens. Various intact rock qualities with blocky and very blocky structure are examined. The results indicate deviations in the rock mass strength predicted by the two approaches, especially when the intact rock strength is low.

1. Introduction

Civil and mining structures involve the disturbance of large volumes of rock masses, which consist of interlocked intact rock pieces separated by various kinds of discontinuities. Such a disturbance is associated with the changes in the force field within the mass. By modelling this prototype rock mass as an equivalent continuous rock mass medium, evolved from a homogenization procedure, this force field can be represented by a Cauchy type stress field. This stress field, by definition, may nowhere overcome the pertinent Cauchy type strength of the rock mass. Any experimental procedure appropriate to define this Cauchy type strength should be applied on a rock mass that contains a statistically adequate number of intact rock pieces and discontinuities. For a sparsely jointed rock mass, this would involve large size testing of the prototype rock mass, which in general may be considered, amongst others, practically unfeasible. Therefore, the most common way to express this Cauchy type strength of the rock masses is through the use of empirical failure criteria. These criteria evaluate the in situ rock mass strength as a fraction of the laboratory defined intact rock strength, according to the rating of the quality of the rock mass. The latter depends on the structure and the surface condition of the discontinuities, and is quantified by the various rock mass classification systems.

The most widely used Cauchy type strength failure criterion is the one presented by Hoek and Brown [1]. This criterion (HB) has been modified since then, and its final form is expressed as [2]

$$\sigma'_1 = \sigma'_3 + \sigma_{ci}\left(m_b\frac{\sigma'_3}{\sigma_{ci}} + s\right)^a \tag{1}$$

where

$$m_b = m_i \exp\left(\frac{GSI-100}{28-14D}\right) \tag{2}$$

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

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right) \tag{4}$$

In the above expressions, σ'_1 and σ'_3 are the major and minor effective Cauchy type stresses at failure, m_i is a constant dependent on rock type, σ_{ci} is the uniaxial compressive strength of an intact rock, D is a disturbance factor, and GSI [3] is the geological strength index, which is the rating of the homonymous rock mass classification system. The latter quantifies the quality of the rock mass according to the structure of the rock mass and the quality of the joint surfaces (Fig. 1). The strength of this continuous rock mass model is by definition isotropic and monotonically non-linearly increasing with the confining stress. Below, for abbreviation, the model and its strength are declared as the GSI model and the GSI strength.

The failure loci of the Hoek–Brown failure criterion for an intact rock can directly be compared with the respective loci

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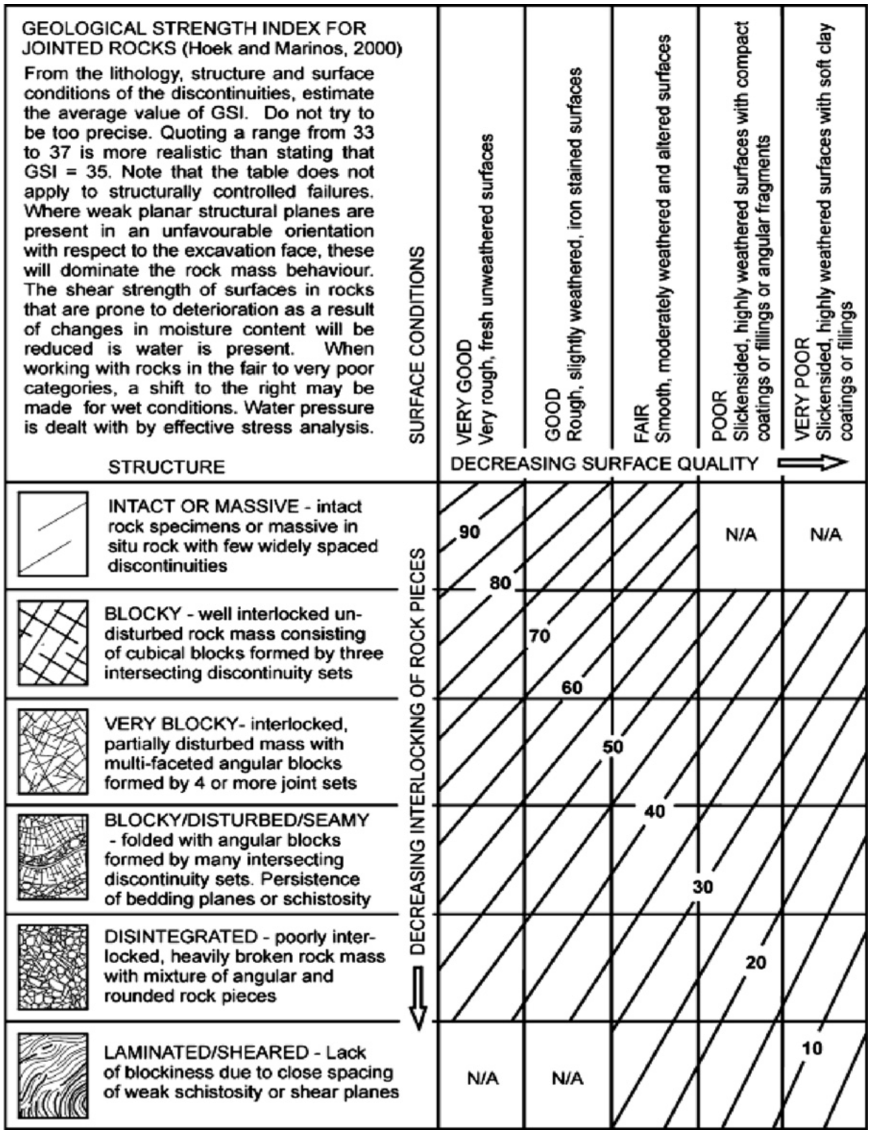

Fig. 1. GSI of rock masses [3].

based on triaxial laboratory tests. However, failure loci (envelopes to Mohr circles) of the jointed rock masses produced by the GSI model have not been adequately confirmed either in the laboratory or in situ, which led Kolymbas [4] to pose scepticism on its realm of applicability and its rationality. Furthermore, Hoek and Marinos [5] acknowledge the need to at least calibrate, if not replace completely some of the empirical methods, such as the GSI model, with numerical tools.

An analytical investigation on the strength of jointed rock masses has already been attempted more than 40 years ago by Jaeger and Cook [6], with the application of the original plane of weakness theory on multiply jointed rock. Recently, this theory has been extended, by Halakatevakis and Sofianos [7], to consider the strength of rock masses, crossed by multiple sets of discontinuities, of varying persistence. There, the rock mass model is composed of intact rock pieces interacting on their contact surfaces, i.e. the joints. The intact rock obeys the HB failure criterion, whereas the discontinuities obey the non-linear Barton–Bandis (BB) [8] failure criterion, expressed by

$$\tau = \sigma_n \tan\left[JRC \log\left(\frac{JCS}{\sigma_n}\right) + \phi_b\right] \tag{5}$$

where τ is the shear stress, σ_n is the normal stress on the joint plane, *JRC* is the joint roughness coefficient, *JCS* is the joint wall compressive strength, and ϕ_b is the basic friction angle.

The limited persistence of the discontinuities is simulated assuming an equivalent shear strength, which includes a percentage of the strength of the original discontinuity and a percentage of the strength of the intact rock.

Because the failure of the jointed rock undergoes complicated processes [9], the extended theory has been applied to experiments made by various authors, which included different failure mechanisms than those predicted by the theory [7]. The extended theory was able to predict adequately the jointed rock strength for a variety of failure mechanisms. Variations were observed when the failure was caused by axial splitting of the intact material or when the failure was due to the formation of kink bands. These failure mechanisms were observed in the experiments for confining pressures less than 1–2 MPa. Thus, the extended theory can be regarded as a reliable tool for an estimation of the jointed rock strength for confining pressures higher than 1–2 MPa and provides an upper bound of this strength for lower confining pressures.

The discrete rock mass model may be declared for an abbreviation as the EPW model, and its strength the EPW strength. Below, an attempt is made to correlate the GSI strength with the analytically derived EPW strength, of the continuous and the discrete models, simulating the same prototype sparsely jointed rock mass.

Table 1
Intact rock parameters.

	Low strength	Moderate strength	High strength
σ_{ci} (MPa)	10	50	100
m_i	5	10	20

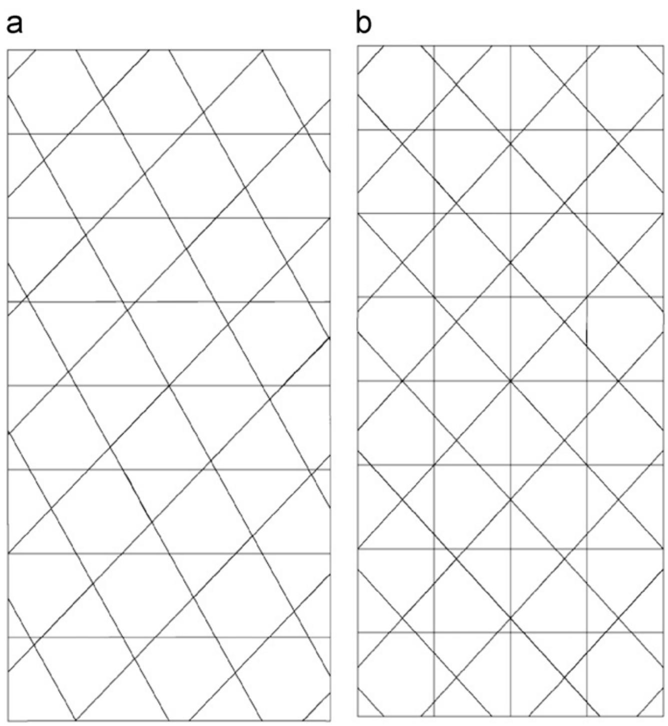

Fig. 2. Blocky (a) and very blocky (b) discrete EPW rock mass models.

Table 2
BB joint properties to the GSI joint quality categories.

Parameter	Joint quality				
	Very good	Good	Fair	Poor	Very poor
JRC	16–20	12–16	8–12	4–8	0–4
JCS/σ_{ci}	0.8–1	0.6–0.8	0.4–0.6	0.2–0.4	0–0.2

2. Equivalent model parameters

Continuous and discrete models have to be attributed pertinent parameters. The strength of the continuous GSI rock mass model depends on the three parameters: σ_{ci}, m_i and GSI. The unconfined compressive strength σ_{ci} of the intact rock and the parameter m_i are intact rock quantitative parameters, which may be easily determined experimentally. Further, the GSI rating is an engineering geological parameter, based on field observations on rock structure and joint surface quality.

The strength of the latter discrete EPW rock mass model depends on the strength parameters for the intact rock σ_{ci} and m_i, as for the continuum model, and on the rock structure and the BB shear strength parameters of the discontinuities. The strength of the intact rock is defined by σ_{ci} and m_i. The range of the values of these parameters is presented in [3]. Here, the strength of the intact rock is assumed to take three representative values, a low, a moderate and a high one, as shown in Table 1. As the intact rock parameters are the same for both models, to compare their strength, the GSI rating of the continuous GSI model has to be related to the rock structure configuration and the shear strength parameters of the discontinuities of the discrete EPW model.

Two rock structure configurations are considered for the sparsely jointed rock mass, which is assumed to be either blocky or very blocky, according to the GSI system. The blocky structure is presented in Fig. 2a, where the three degrees of freedom have been maintained. The very blocky structure is presented in Fig. 2b, where the four degrees of freedom have been maintained. The two rock structures considered may contain either persistent or non-persistent joints. The GSI system does not consider joint persistence explicitly [10], and therefore the strength of the GSI model will remain unchanged for varying degrees of joint persistence. On the other hand, the strength of the EPW depends on joint persistence. Therefore, differences arising due to the abatement of joint persistence, from 100% to 50%, are investigated for the EPW rock mass model.

The joint quality according to the GSI system varies from very good to very poor. These quality characteristics for the EPW model have to be assigned pertinent joint shear strength parameters ϕ_b, *JRC* and *JCS*. The former varies [11] between 23° and 40° with most probable value of 30°; the latter value is assigned, as it is not considered explicitly by the GSI system. The *JRC* parameter according to [12] ranges 0–20, according to the roughness of the joint surface. The *JCS* parameter equals the strength of the joint walls and depends on their degree of weathering. Therefore, it ranges between the uniaxial compressive strength of the intact rock, for fresh unweathered surfaces, and practically zero, which holds for extremely weathered and eroded surfaces. According to the GSI system, there is only one combination for joint roughness and weathering in each joint quality category. Thus, scaling the above BB joint parameter ranges on the GSI joint quality categories provides the distribution of the *JRC* and *JCS* values, shown in Table 2. The joint category "Very Poor" applies mostly for joints with filling material, and

Table 3
Equivalent GSI values.

Rock mass structure	Joints quality							
	Very good		Good		Fair		Poor	
BLOCKY	$JRC=18$ $JCS/\sigma_{ci}=0.9$	GSI=75	$JRC=14$ $JCS/\sigma_{ci}=0.7$	GSI=65	$JRC=10$ $JCS/\sigma_{ci}=0.5$	GSI=55	$JRC=6$ $JCS/\sigma_{ci}=0.3$	GSI=45
VERY BLOCKY	$JRC=18$ $JCS/\sigma_{ci}=0.9$	GSI=65	$JRC=14$ $JCS/\sigma_{ci}=0.7$	GSI=55	$JRC=10$ $JCS/\sigma_{ci}=0.5$	GSI=45	$JRC=6$ $JCS/\sigma_{ci}=0.3$	GSI=35

therefore is not further considered. For the remaining four categories, the mean value of the parameters *JRC* and *JCS* is attributed to each one. Hence, for each pair of rock structure (i.e. Blocky or Very Blocky) and joint quality (i.e. Very Good to Poor), shown in Fig. 1, parameters *JCS* and *JRC* for the Barton–Bandis (BB) criterion, and parameter GSI for the Hoek–Brown (HB) criterion, are attributed, as shown in Table 3.

3. Strength anisotropy

To compare the discrete EPW model with its equivalent continuous GSI model, both models are examined for all combinations of their properties. Three intact rock properties, two rock structural configurations with two types of persistence, and four joint quality categories of the prototype rock mass are simulated by both types of models. The strength of these models in a confining stress regime up to 8 MPa is evaluated, to compare the performance of equivalent configurations.

3.1. Model formulation

A blocky rock mass, which contains three joint sets with the same engineering properties, is modelled initially as an EPW rock mass. The initial dip angle of the joint sets is 0°, 45° and 120°. The dip angle of the joint system is defined as β. The joint system is rotated by 180° and the EPW rock mass strength is calculated for every 5° of rotation. This procedure is repeated assuming various joint persistence combinations starting with being fully persistent. Then, one of the joint sets is assumed as 50% persistent; the next simulation is done assuming two of the joint sets being 50% persistent, and finally all joint sets are taken as 50% persistent. Further, for the equivalent continuous GSI rock mass, the HB failure criterion is employed, and its strength is evaluated. Thus, for each discrete EPW model and its equivalent continuous GSI one, the strength is evaluated at various confining stress levels and dip angles.

The same methodology is followed for the case of a very blocky rock mass. The EPW theory is applied for four joint sets with equal engineering properties. The initial dip angle of the joint sets is 0°, 45°, 90°, and 120°. The joint system is rotated by 180° and the EPW rock mass strength is calculated for every 5° of rotation. This procedure is repeated similarly, assuming various joint persistence combinations. All joints initially are fully persistent. Then, sequentially, one, two, three and four of the joint sets are assumed as being 50% persistent.

3.2. Anisotropy of strength

In Figs. 3 and 4, the failure strength of the discrete EPW and the continuous GSI models with low intact rock strength σ_{ci} and with either blocky (GSI=75) or very blocky fully persistent joint structure (GSI=65), respectively, is drawn as a function of dip angle β of the joint system, for four values of confining stress. It may be observed that the failure strength of the discrete EPW rock mass is fluctuating about a central value, and may practically be considered as quasi isotropic, especially for the case of the very blocky structure. On the other hand, the strength provided by the HB failure criterion is by definition isotropic, i.e. independent of the joint sets dip angle. For the case of blocky structure and confining stress up to 2 MPa, the GSI model isotropic strength (locus) can be regarded as a mean value to an anisotropic strength provided by the EPW model. For higher values of confining stress, the EPW model predicts higher anisotropic strength. For the very blocky structure, the EPW model predicts higher anisotropic strength for all values of the confining stress.

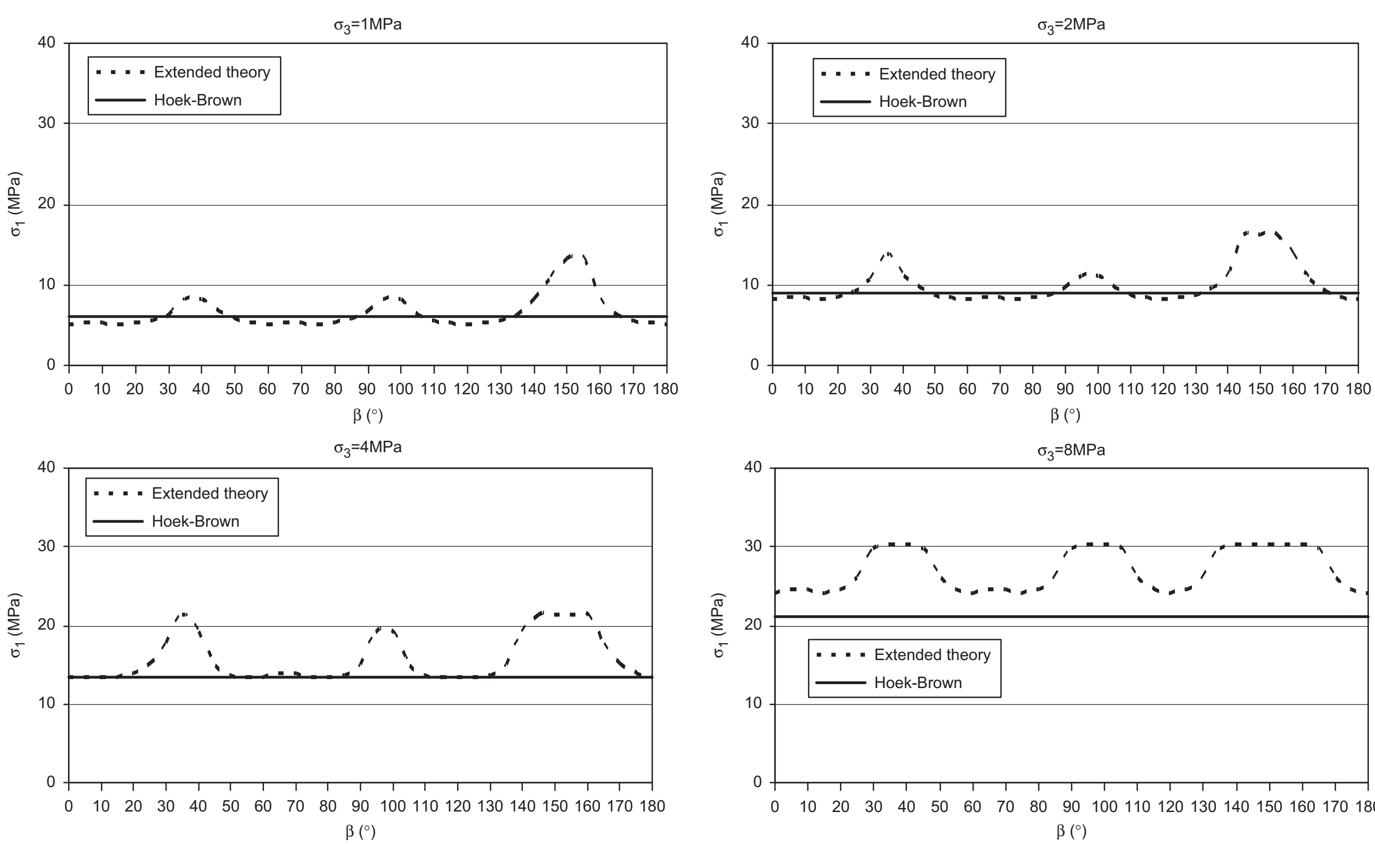

Fig. 3. Anisotropic strength, at various confining stress levels, of EPW and GSI rock mass models, with low intact rock strength and blocky structure.

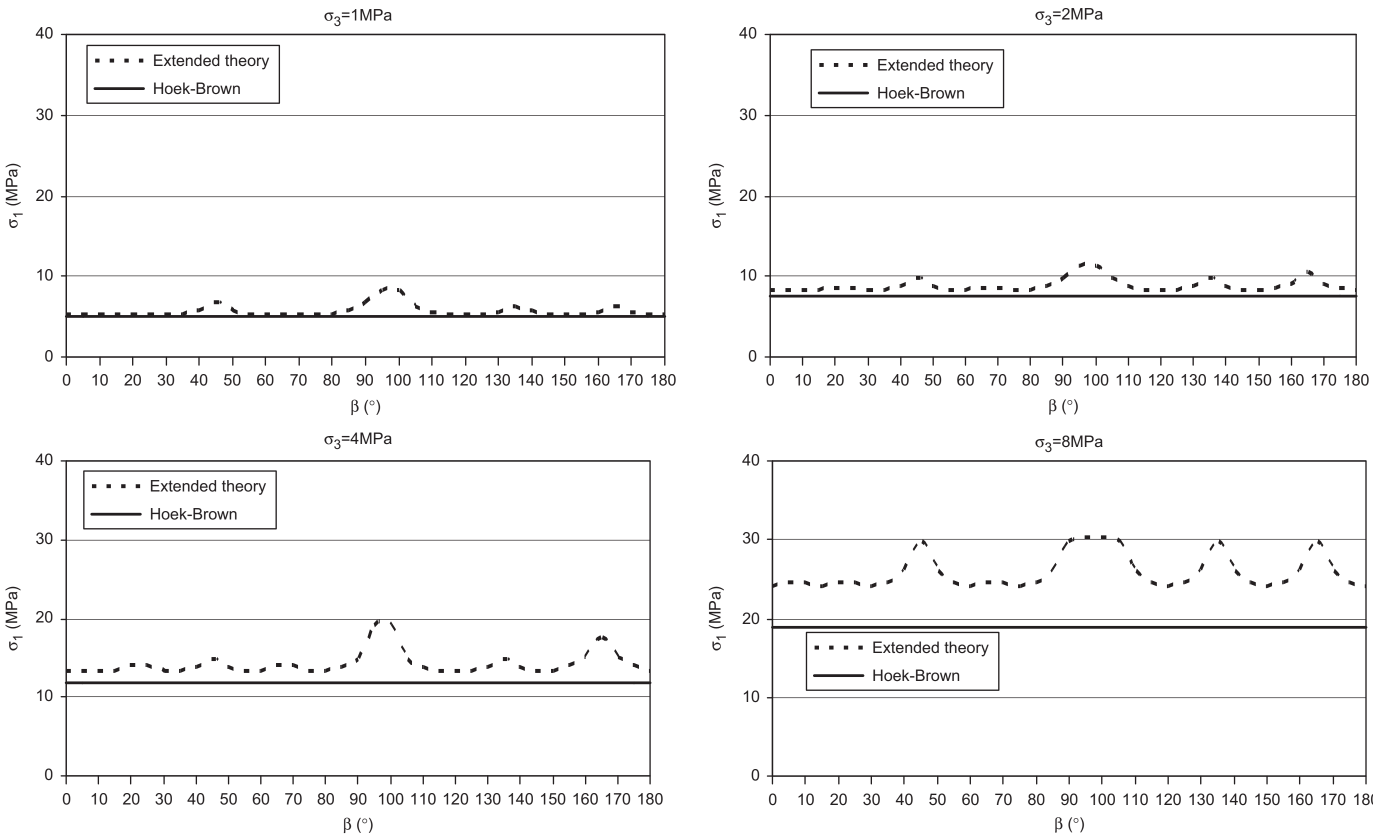

Fig. 4. Anisotropic strength, at various confining stress levels, of EPW and GSI rock mass models, with low intact rock strength and very blocky structure.

Figs. 5 and 6 show the strength of discrete EPW and continuous GSI models with moderate intact rock strength σ_{ci} and with either blocky (GSI=75) or very blocky (GSI=65) fully persistent joint sets structure, respectively. It may be observed that the strength of the discrete EPW rock is fluctuating about a mean value and may be considered as quite isotropic, especially for the case of very blocky structure. On the other hand, as stated previously, the strength provided by the GSI model is independent of the joint sets dip angle. For both rock structure types, the strength of the GSI model can be regarded as a mean value of the varying with dip strength of the pertinent discrete EPW models.

Figs. 7 and 8 show the strength of discrete EPW and continuous GSI models with high intact rock strength, σ_{ci} and with either blocky (GSI=75) or very blocky (GSI=65) fully persistent joint sets structure, respectively. It may be observed that the strength of the discrete EPW rock models is fluctuating about a mean value and may be considered as quite isotropic, especially for the case of very blocky structure. On the other hand, as stated previously, the strength provided by the GSI model is independent of the joints dip. For both rock structure types, the strength of the GSI model can be regarded as an upper limit to the varying with dip strength of the pertinent discrete EPW models.

4. Strength of equivalent isotropic rock mass

As shown in the previous section, the strength of blocky and very blocky discrete EPW rock models fluctuates with joint sets dip angle about a mean value. The HB failure criterion is applied to isotropic jointed rocks, so for rock masses with blocky and very blocky structures it is taking into account an equivalent isotropic strength. An equivalent isotropic strength for the discrete EPW rock models can be defined by the estimation of the mean strength, i.e. the average strength over all joint sets dip angle values. This average strength is computed at an interval of 5° of joint system rotation. This interval is small enough, so it is regarded as satisfactory for the computation of the mean strength. Thus, this mean value of the strength of the EPW models is evaluated at all levels of the considered confining stress. Following this procedure, isotropic EPW model failure strength loci (failure locus of the Mohr circles) within the considered geotechnical engineering confining stress range, are drawn for the discrete rock models. These are then compared to the failure strength loci evaluated for the equivalent continuous GSI rock models.

4.1. Low intact rock strength

Initially the models are considered to have low intact rock strength. For each discrete EPW rock mass model, the mean isotropic strength is calculated at each value of confining stress. This procedure is repeated for four values of confining stress and the isotropic EPW failure locus (failure criterion), is drawn. This failure locus may thus be compared with that of the equivalent GSI continuous model, i.e. the HB failure criterion.

Fig. 9 presents the failure loci provided by the above approaches, for the case of low intact rock strength with blocky structure (GSI=75, 65, 55, 45) with four joint persistence combinations. The EPW rock mass model in all cases predicts higher strength than that of the GSI model. Further, the joint sets persistence affects the failure loci estimated by the EPW rock mass models. When all joints are assumed to be non-persistent, the EPW isotropic failure locus approaches that for the intact rock. Furthermore, the EPW isotropic failure locus is almost linear, in contrast to that of the GSI one.

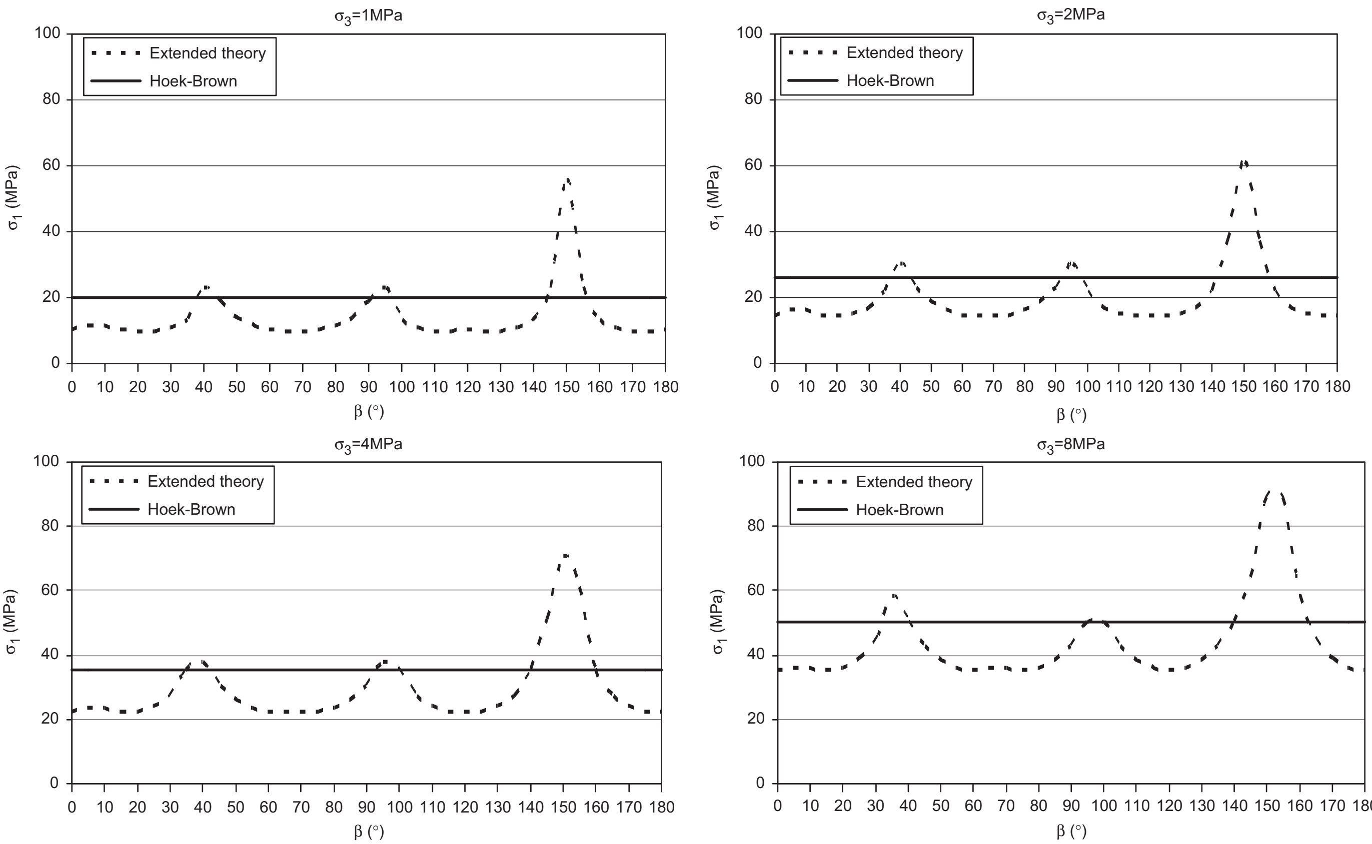

Fig. 5. Anisotropic strength, at various confining stress levels, of EPW and GSI rock mass models, with moderate intact rock strength and blocky structure.

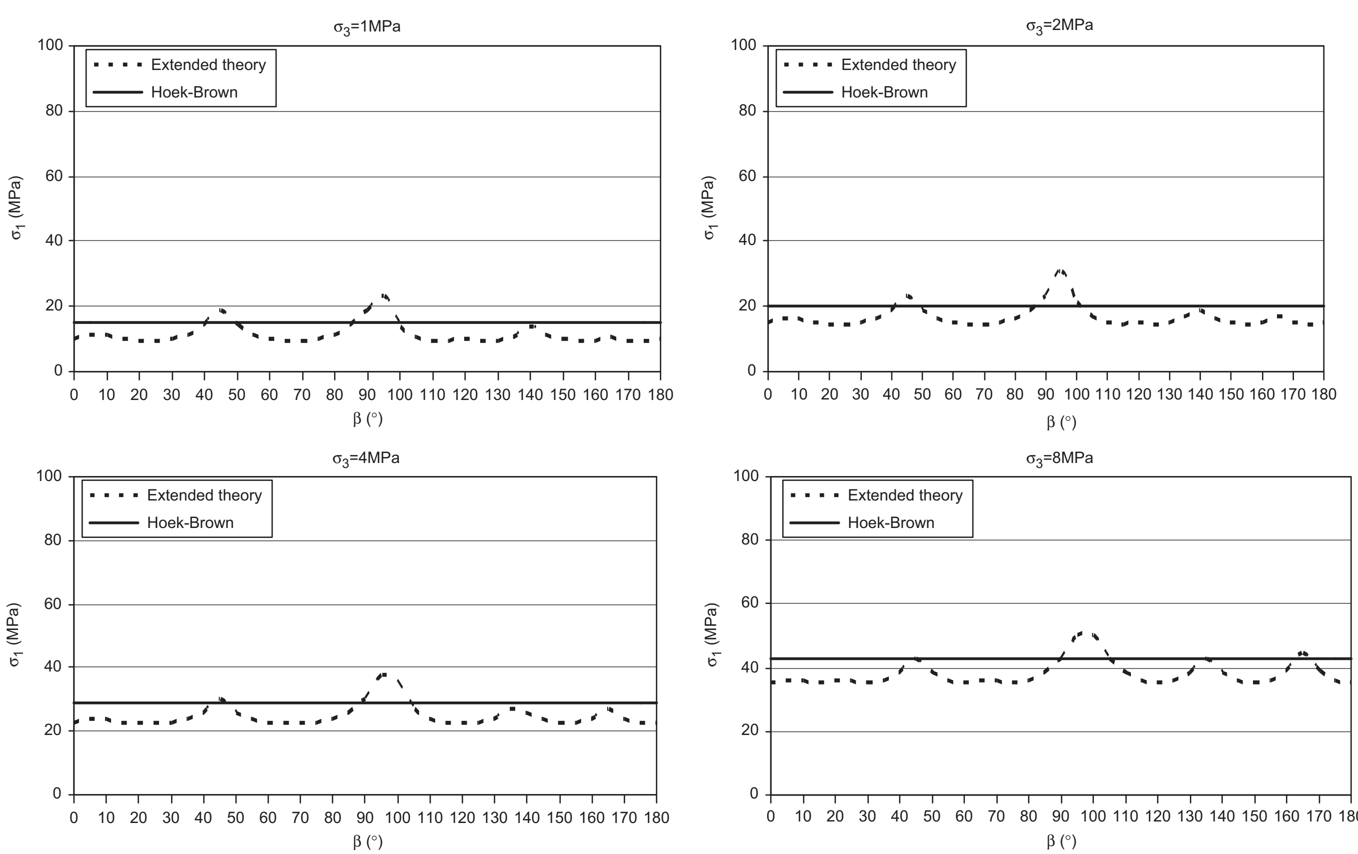

Fig. 6. Anisotropic strength, at various confining stress levels, of EPW and GSI rock mass models, with moderate intact rock strength and very blocky structure.

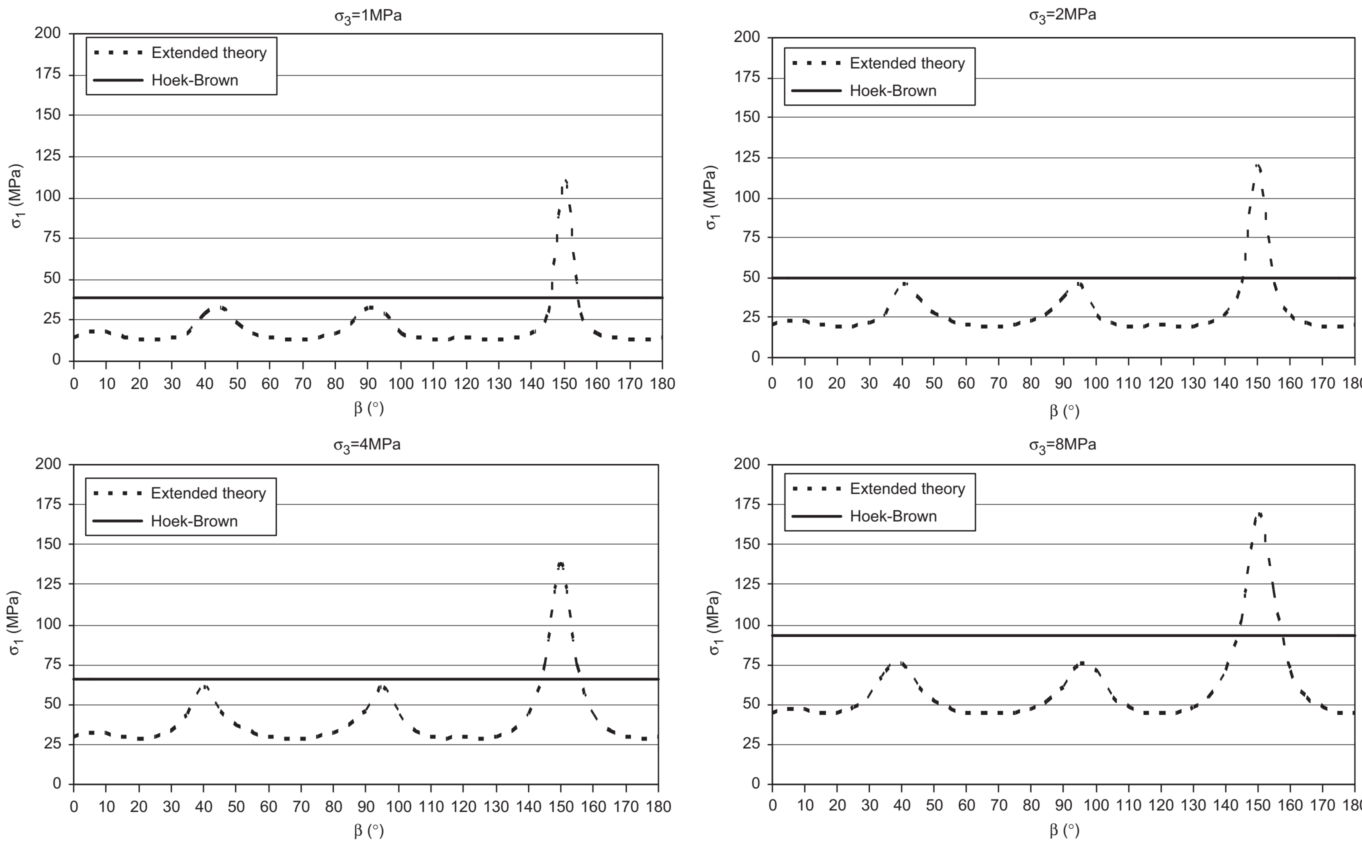

Fig. 7. Anisotropic strength, at various confining stress levels, of EPW and GSI rock mass models, with high intact rock strength and blocky structure.

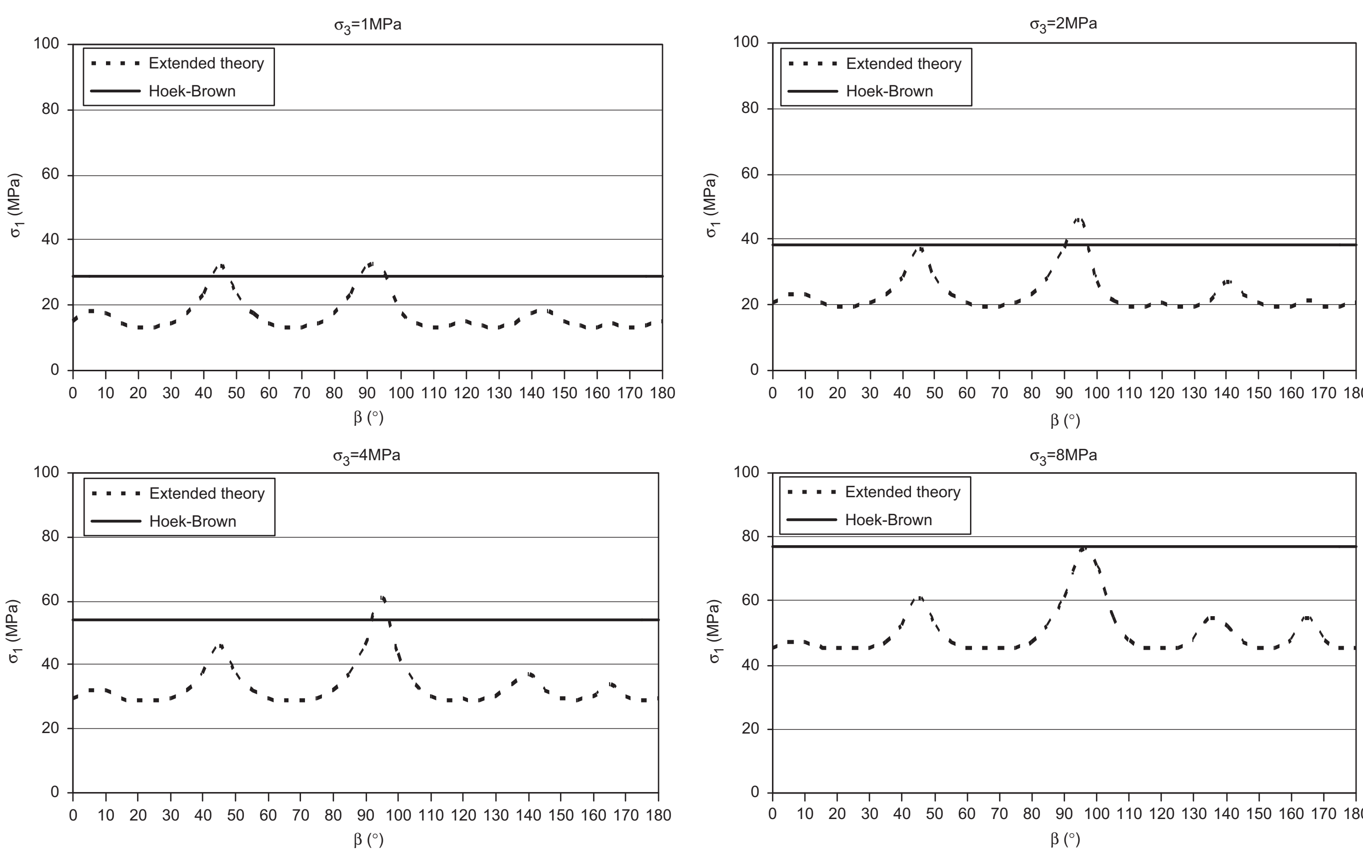

Fig. 8. Anisotropic strength, at various confining stress levels, of EPW and GSI rock mass models, with high intact rock strength and very blocky structure.

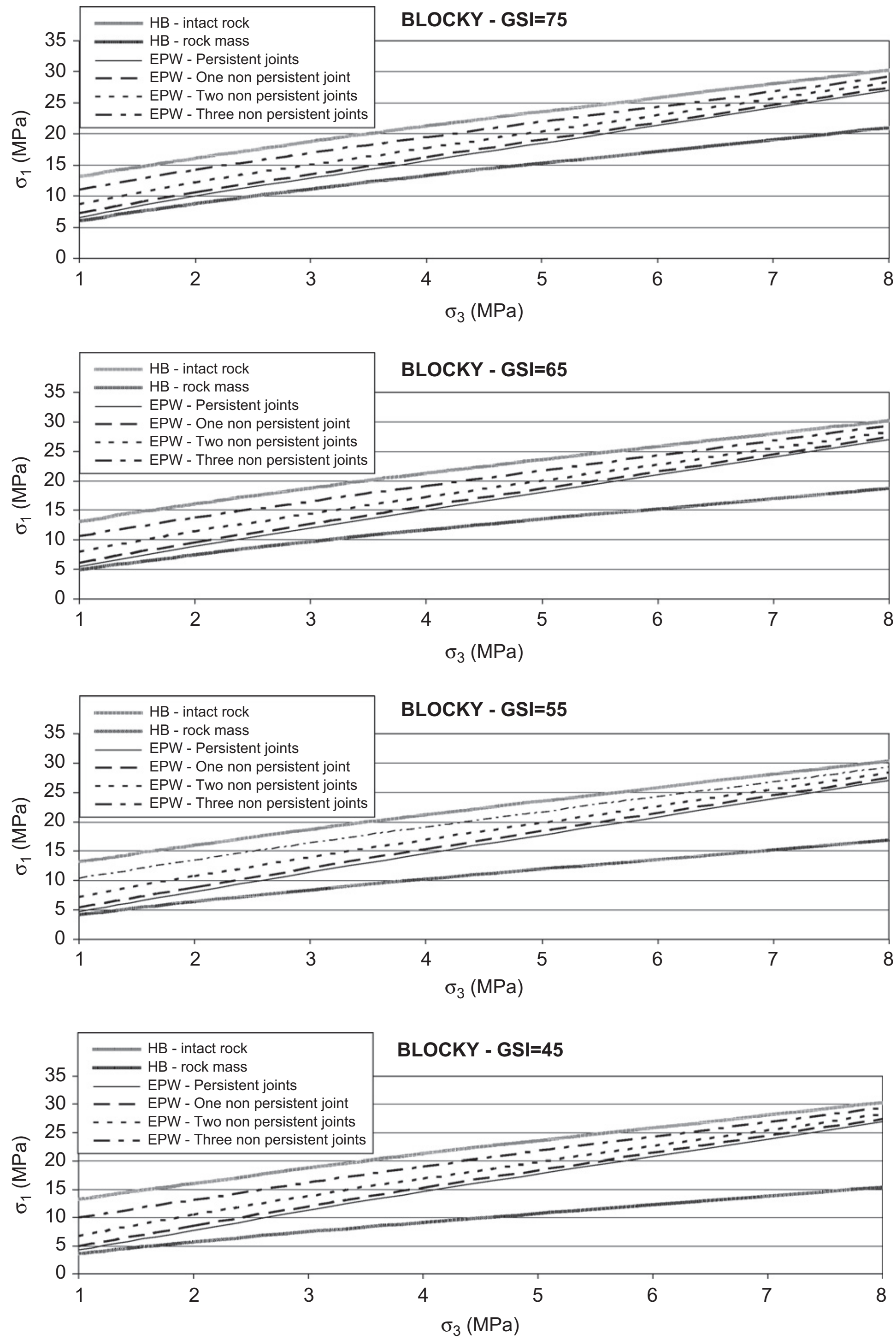

Fig. 9. Failure loci for models with low intact rock strength and blocky structure.

A possible explanation for the linear form of the EPW failure locus and the high divergence between the two approaches for high values of confining stress is the low value of uniaxial compressive strength of the intact rock (σ_{ci}=10 MPa) and, in turn, the low values of the joint wall strength (*JCS*). As the value of the confining stress rises, the normal stress acting on the joint plane approaches the value of *JCS*, and so the effect of joint roughness on the rock mass strength is minimized. This phenomenon is simulated with the EPW theory, but is not with the HB failure criterion.

Fig. 10 presents the isotropic EPW model failure loci and the GSI model one, for the case of low intact rock strength with very blocky structure (GSI=65, 55, 45, 35) and five joint persistence combinations. The EPW model in all cases predicts higher strength than that of the GSI one. Further, the joint sets persistence affects the failure loci evaluated by the EPW models. When all joints are assumed to be non-persistent the isotropic EPW failure locus approaches that of the intact rock. Furthermore, this isotropic failure locus is almost linear, in contrast to that of the HB failure criterion. The same explanation for the linear form of the EPW failure locus and the high divergence between the loci of the two models, for high values of confining stress, is given, as for the case of the blocky rock mass.

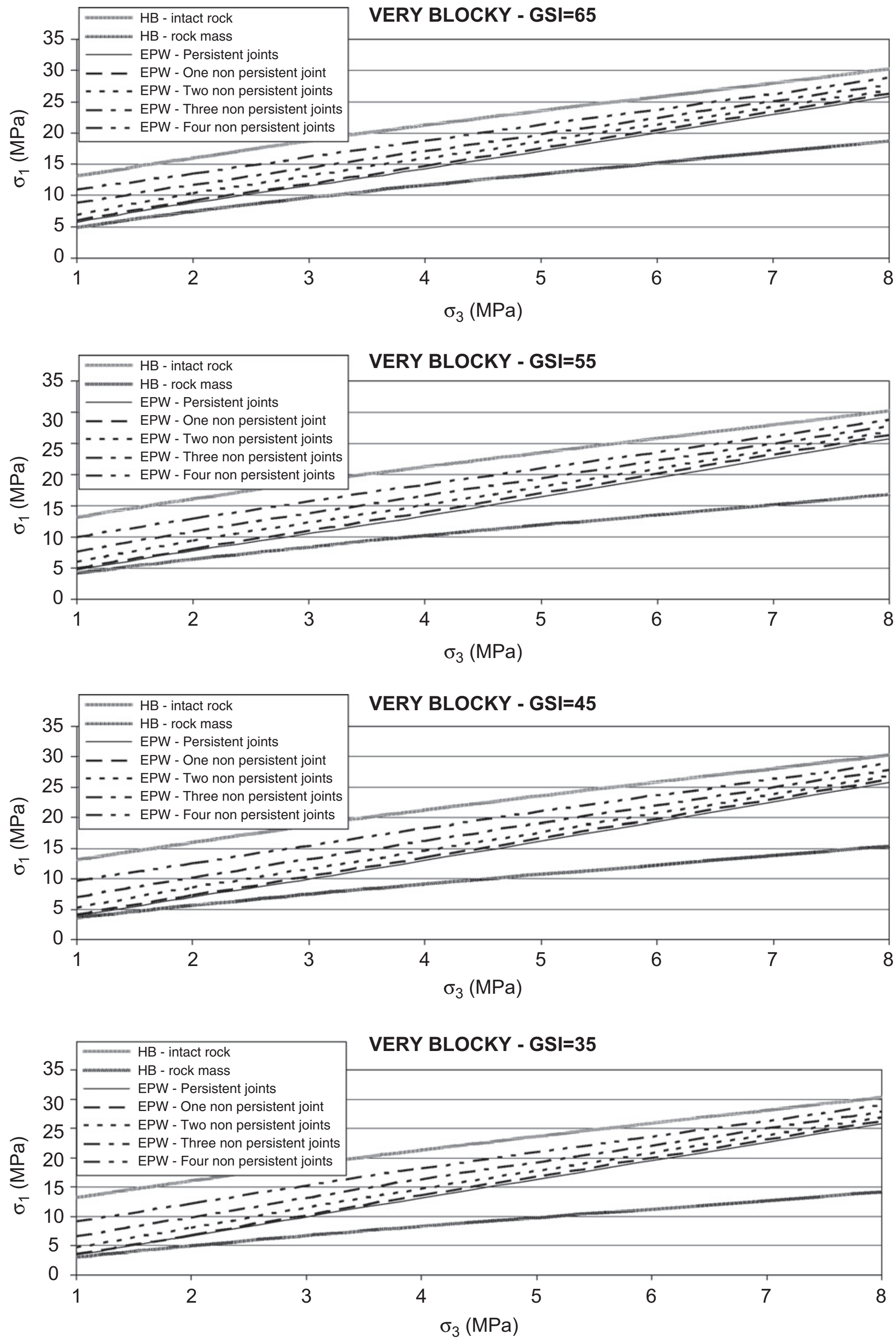

Fig. 10. Failure loci for models with low intact rock strength and very blocky structure.

4.2. Moderate intact rock strength

Next, the intact rock strength is considered to be moderate. For each discrete EPW model at each value of confining stress, the mean isotropic EPW strength, is calculated. This procedure is repeated for four values of confining stress and the isotropic EPW rock mass model failure locus is drawn. The equivalent GSI model failure locus may thus be compared to the discrete EPW one.

Fig. 11 presents the failure loci provided by the above approaches for the case of moderate intact rock strength with blocky structure (GSI=75, 65, 55, 45) and four joint persistence combinations. The EPW model predicts lower strength than that of the GSI model when all joints are persistent. When one of the joints is non-persistent, the equivalent isotropic strength of the EPW model is approximately equal to that predicted by the GSI model. When all joints are assumed to be non-persistent, the isotropic failure locus approaches that of the intact rock. The equivalent isotropic failure locus appears to be non-linear, as compared to the GSI model failure loci, in contrast to the previous case of low intact rock strength.

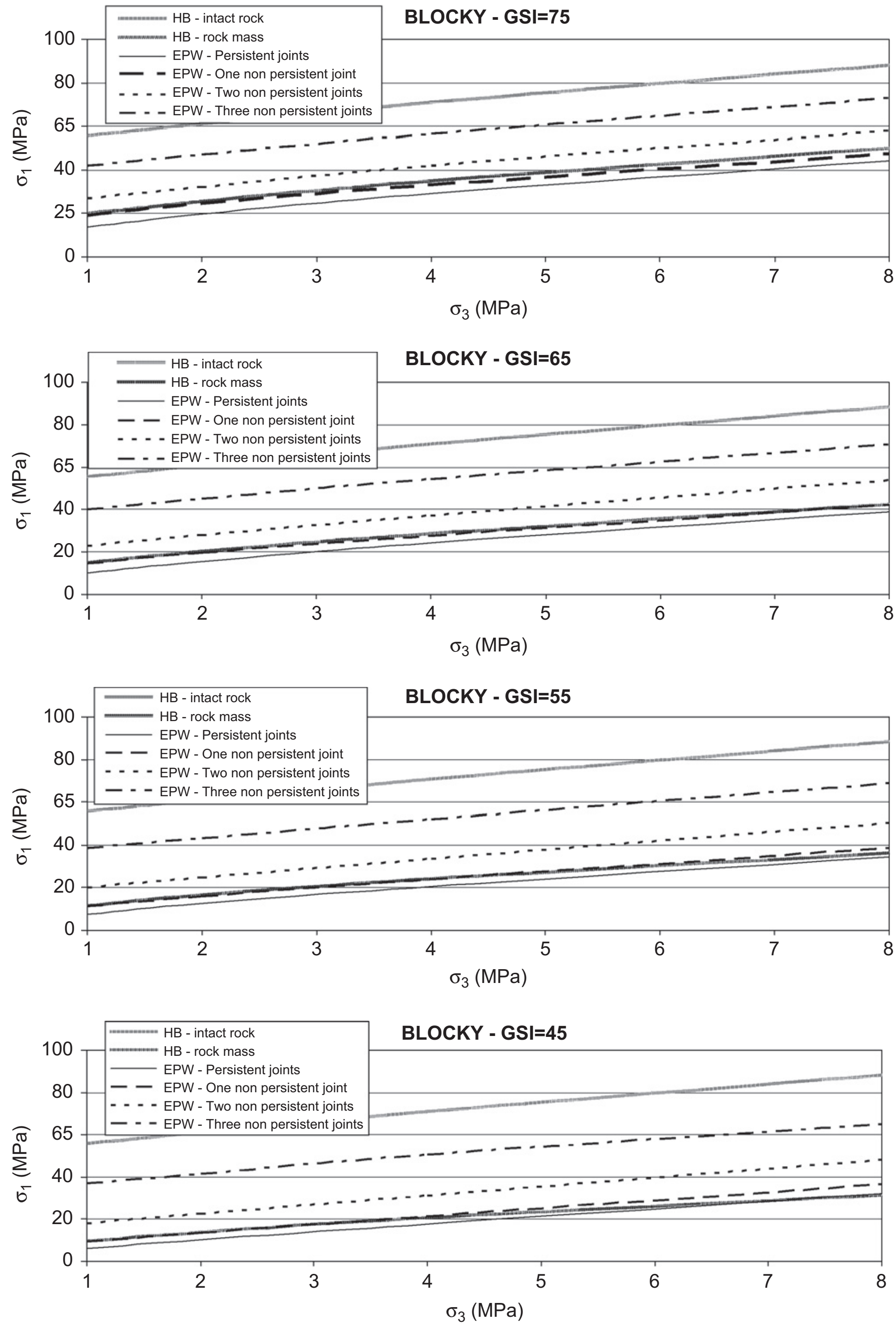

Fig. 11. Failure loci for models with moderate intact rock strength and blocky structure.

Fig. 12 presents the failure loci provided by the above approaches for the case of moderate intact rock strength with very blocky structure (GSI=65, 55, 45, 35) and five joint persistence combinations. The EPW rock mass model predicts lower strength than that of the HB failure criterion when all joints are persistent. The failure locus, predicted by the equivalent GSI model, lies between the isotropic EPW failure loci for rock structures containing one and two non-persistent joints. When all joints are assumed to be non-persistent, the equivalent isotropic failure locus approaches that for the intact rock. The equivalent isotropic failure locus appears to be non-linear, as compared to the GSI model failure loci, in contrast to the previous case of low intact rock strength.

4.3. High intact rock strength

Finally, the intact rock strength is considered to be high. For each discrete EPW model at each value of confining stress, the mean isotropic EPW strength, is calculated. This procedure is

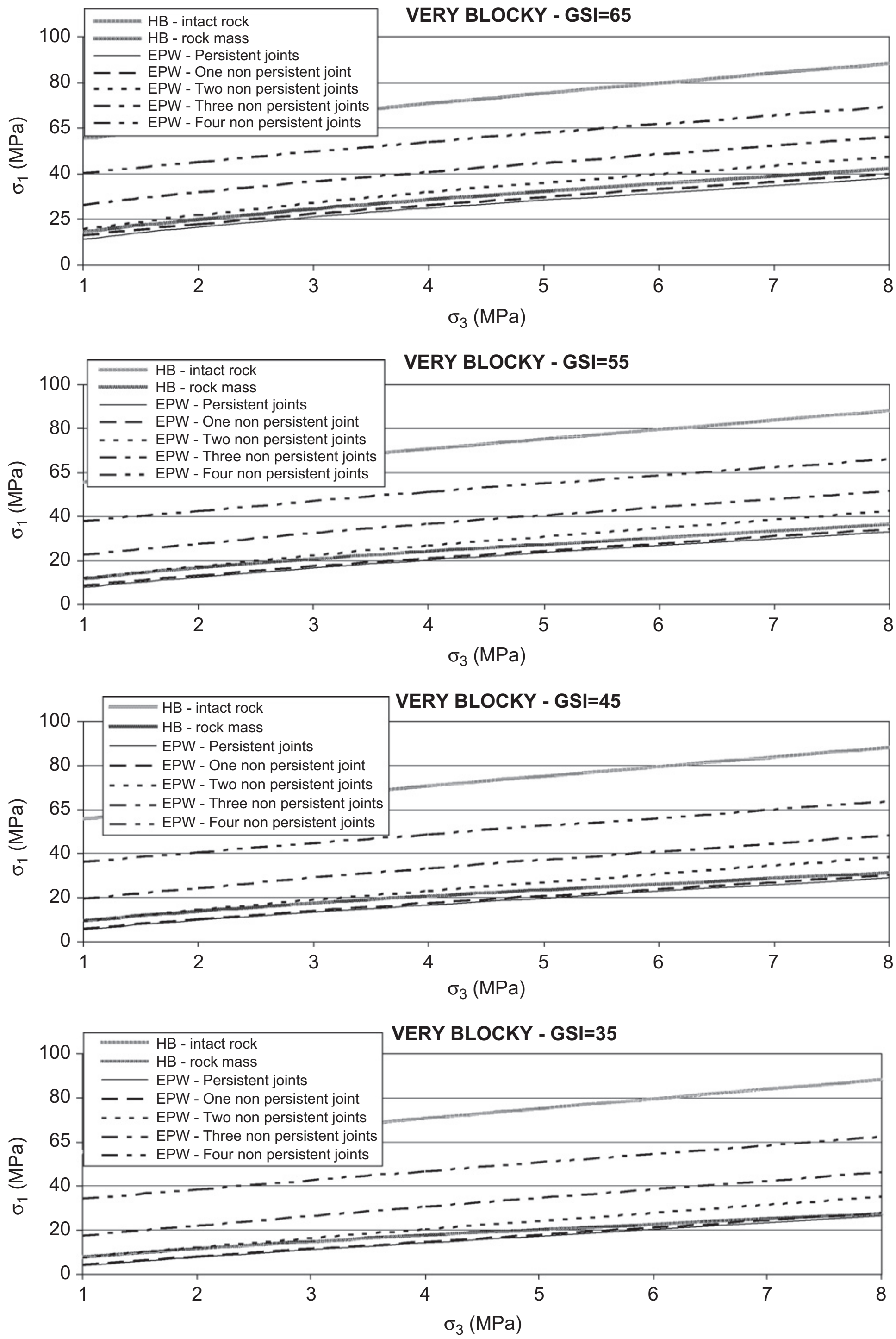

Fig. 12. Failure loci for models with moderate intact rock strength and very blocky structure.

repeated for four values of confining stress and the isotropic EPW failure locus is drawn. The failure locus for the equivalent GSI model may thus be compared to the discrete EPW model one.

In Fig. 13, the failure loci provided by the above approaches are presented for the case of a rock mass, with high intact rock strength, blocky structure (GSI=75, 65, 55, 45) and four joint persistence combinations. The EPW model predicts lower strength than that of the GSI model, when all joints are persistent. The failure locus of the equivalent GSI rock mass model, given by the HB failure criterion, lies between the isotropic EPW failure loci for rock structures containing one and two non-persistent joints. When all joints are assumed to be non-persistent, the isotropic EPW failure locus approaches that of the intact rock.

In Fig. 14, the failure loci evaluated by the above approaches are presented for the rock mass with high intact rock strength, very blocky structure (GSI=65, 55, 45, 35) and five joint persistence combinations. The EPW model predicts lower strength than that of the GSI model, when all joints are persistent. The failure locus, of the equivalent GSI model, lies between the isotropic EPW failure loci for two and three non-persistent joints. When all joints are assumed to be non-persistent, the equivalent isotropic failure locus approaches that of the intact rock.

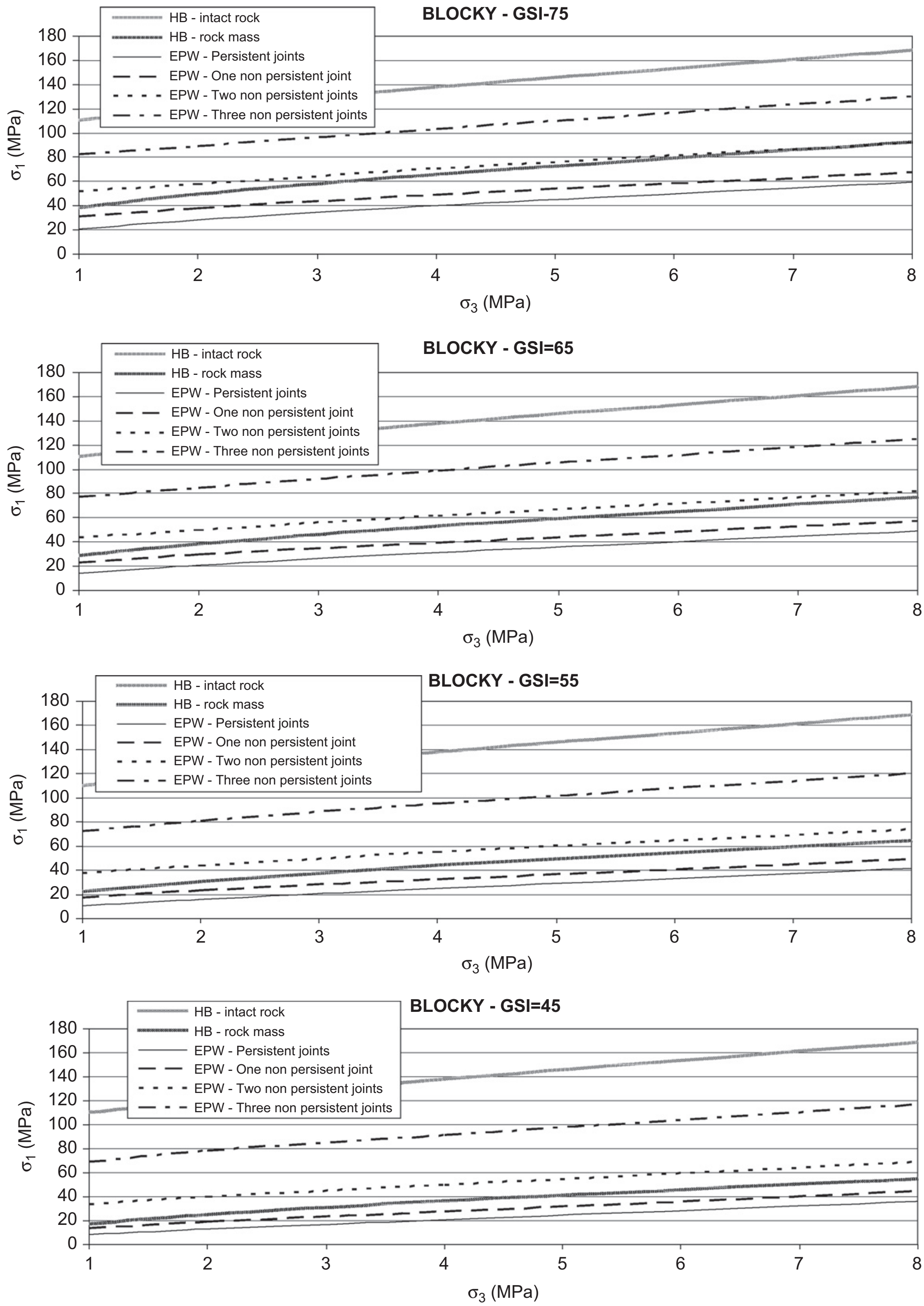

Fig. 13. Failure loci for models with high intact rock strength and blocky structure.

5. Discussion–Conclusions

Difficulties in sampling and testing of jointed rock specimens have led to the development of empirical failure criteria for the estimation of the rock mass strength. The Hoek–Brown failure criterion is the most widely accepted one, but failure loci of this criterion, for jointed rock masses, have not been adequately confirmed either in the laboratory or in situ. This fact raises issues of uncertainty about its applicability to jointed rocks. Here, an analytical investigation of this criterion is attempted, with the application of the extended plane of weakness (EPW) theory on multiply jointed rock. The intact rock obeys the HB failure

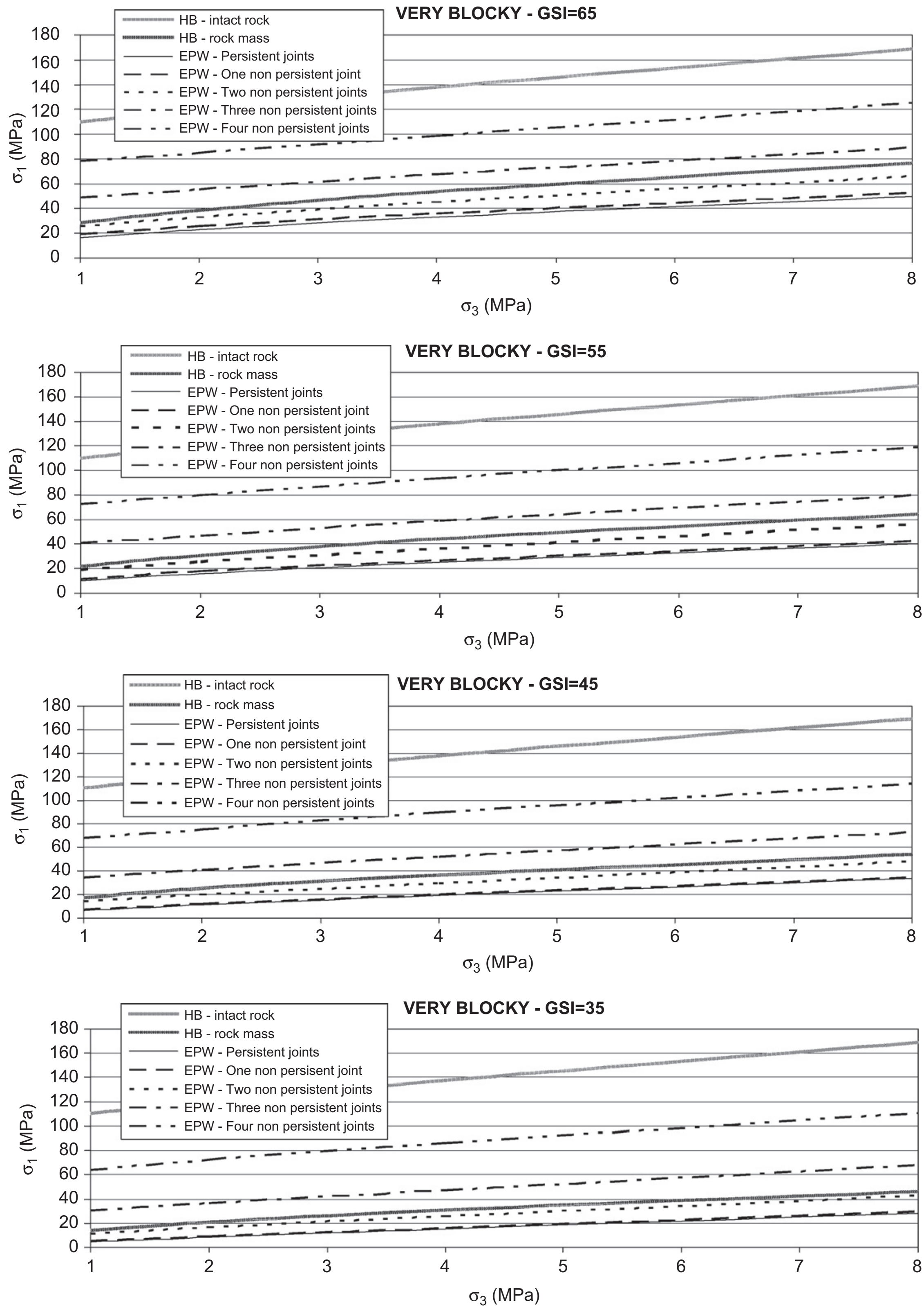

Fig. 14. Failure loci for models with high intact rock strength and very blocky structure.

criterion, whereas the discontinuities obey the non-linear Barton–Bandis (BB) failure criterion. This criterion has been chosen, as it predicts the non-linear behaviour of the joints, with relatively easily estimated input parameters. Nevertheless, to simulate any more complex behaviour of the joints, a more sophisticated joint failure criterion may be easily adopted, to further improve the strength prediction capability of the EPW model.

Two rock structure configurations are considered for the sparsely jointed rock mass, which is assumed to be either blocky or very blocky. The two rock structures considered may contain either persistent or non-persistent joints. An equivalent GSI model is produced by scaling the BB parameters to the GSI joint surface quality categories. This methodology allows a direct comparison between the EPW theory and the HB failure criterion, via their failure loci. The EPW model failure envelopes exhibit variations compared to those obtained by the HB failure criterion. These variations are due to the limited joint persistence and when the low intact rock strength is considered.

Initially, the EPW theory is applied to a jointed rock mass of low intact rock strength and blocky to very blocky structures. In both cases, the EPW theory predicts higher strength than that of the HB failure criterion. Also, the joints persistence affects the failure loci estimated by the EPW theory. When all joints are assumed to be non-persistent, the equivalent isotropic failure locus approaches that of the intact rock. The application of the EPW theory to a blocky and a very blocky rock mass with moderate to high intact rock strength leads to different observations. When all joints are persistent, the EPW theory predicts lower strength than that of the HB failure criterion. The strength predicted by the EPW theory approaches that provided by the HB failure criterion when some of the joints are non-persistent. When all joints are assumed to be non-persistent, the equivalent isotropic failure locus approaches that of the intact rock. According to these, it is obvious that the joint persistence is a dominant factor affecting jointed rock strength. The effect of joint persistence is simulated by the EPW theory, but not with the HB criterion, as GSI does not consider joint persistence explicitly.

The failure locus produced by the EPW theory is non-linear, as in the case of the HB failure criterion. However, when rock mass with low intact rock strength is considered, the equivalent isotropic failure locus is almost linear, in contrast to that of the HB failure criterion. A possible explanation, for the linear form of the EPW theory failure loci and the high divergence between the two approaches for high values of confining stress, is the low value of uniaxial compressive strength of the intact rock and, in turn, of the joint wall strength *JCS*. As the value of the confining stress increases, the normal stress acting on the joint plane approaches the value of *JCS*, and so the effect of joints in the rock mass strength is minimized. This phenomenon is simulated with the EPW theory, but not with the HB failure criterion.

The EPW theory can be applied by using different values of joint basic angle of friction. In the methodology presented here, the basic friction angle does not affect the equivalent GSI value. It is considered as a property depending on rock type and a mean value equal to 30° is selected for the calculations. Increase in the friction angle will increase the strength of joints and, in turn, the strength of rock mass. Oppositely, GSI does not consider explicitly this engineering parameter; so, according to the HB failure criterion, a higher value of joint basic angle of friction will have no impact on jointed rock strength.

The usually employed HB failure criterion is independent of the intermediate effective principal stress σ_2'. The EPW theory is also applied in two dimensions, ignoring the intermediate effective principal stress. Therefore, it can be used for the estimation of jointed rock strength when the joint set strikes parallel to the σ_2' direction. When these conditions do not exist, the intermediate principal stress will tend to increase jointed rock strength, as the confining effect to the joints will be larger.

The EPW model requires much more data for the estimation of jointed rock strength than the HB model does. The engineering properties of intact rock are common in both methods. The HB criterion, in order to take into account the presence of the discontinuities, uses the GSI system which is based on field observations, and combines the structure of jointed rock and the quality of joint surfaces, in an oversimplifying way. Hoek and Marinos [5] quote that "it is simply not possible, within the constraints of a classification system based on a limited number of estimated input parameters, to capture the actual behaviour of heterogeneous rock masses". The EPW theory does not suffer from such limitations. Nevertheless, it requires much more effort and quantitative data about joint roughness, strength, friction angle and persistence, for each joint set to improve the prediction capability of the strength of the rock mass.

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