Review of current rock mass characterization practices

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SUMMARY: Modelling of underground structures requires the determination of ground parameters. This entails a plethora of uncertainties in mechanical and spatial aspects. In most cases the parameters have to be adjusted to the different scale between laboratory samples and real dimensions. As direct modelling of all features of complex rock masses commonly is not possible due to the limitations of the calculation method, the ground model often is simplified. The simplification ranges from simply increasing the block size to completely "homogenizing" the rock mass by "smearing" the discontinuities into a continuum. Various simple "upscaling" methods or empirical procedures are used to arrive at the properties of the rock mass. This paper critically reviews the procedures used in upscaling and modelling practise and examines the effects of simplification with the help of numerical simulations. Five reference cases will be calculated taking different classification systems into account, with the intent of showing the variation in results.

1 INTRODUCTION

In rock engineering and design one important challenge is converting the plethora of ground parameters estimated by field mapping, core drilling and laboratory testing into the underground structure model. Very often those parameters have to be adjusted to the different scale of the laboratory sample size and real dimensions. As direct modelling of all features of complex rock masses is normally not possible due to the limitations of the software, simplifications have to be made. Result should be one parameter set which on the one hand still represents a more realistic ground model, therefore not too much information should be lost and on the other hand a program still can work with.

It is common practice to use empirical classification systems as a design method, although it is generally known that these systems often work with highly simplified parameters. When simplifying the model itself by smearing parameters over the complete model and hereby creating a homogenous ground it is obvious that often the failure mode is misconstrued for homogenous ground. Such a model is not able to show e.g. block kinematics, hence leading to a completely wrong design.

Singh & Goel (1999) suggest that it is better to assign a range of ratings for each parameter. "Design experience suggests that average of rock mass ratings (RMR, GSI, RMi, etc.) be considered in the design of support systems."

This paper shows the effects of different degrees of simplification of the model in a parametric study by using the classification systems RMR (Bieniawski, 1989), RMI from (Palmström, 1996) and GSI (Hoek, 2000). The rock mass quality Q-system (Barton et al, 1974) is not considered in this paper as it's not applicable to numerical software. The variation in results makes the difficulty of verifying the correct result clear and depicts the problem discussed in this paper clearly.

2 SIMPLIFICATIONS MADE BY CLASSIFICATION SYSTEMS

2.1 *General*

The fact that classification systems are based on specific experience of course leads to the instance they do not fit to all conditions, hence requires wariness in their application. There again it is up to the designer to decide which parameters for which cases are accurate, hence experience is the keyword and wrong assessment is often the result.

Different parameters can produce the same rating although the conditions completely differ from one another. This may lead to a significant loss in information in the calculation results. It applies in particular when assessing failure modes. The question arises, why we determine a number of rock and joint parameters, if we do not use them in the design process.

Table 1 clearly depicts by means of some exemplary parameters the loss of information during characterization and the different approaches to one parameter set. E.g. not one classification system discussed in this paper takes the scale of the tunnel radius into account, although a 3m diameter tunnel obviously behaves differently than one of double size in the same surrounding ground conditions.

Table 1. Consideration of key parameters in rock mass classification systems

	Structure	*Joint Orientation*	*Joint Strength*	*Tunnel radius*
RMR	RQD, spacing of joints	Adjustment by joint orientation	Condition of joint (roughness, seperation, filling)	-
RMI	Block volume or number of joints	-	Joint condition factor (roughness, alteration)	-
GSI	Blockiness	-	Condition of discontinuity surface	-

One reference case calculated with different “upscaling” methods and classification systems will lead to several different results, some not even showing the real failure mode as will be shown later on more in detail.

2.2 *Difficulties during measuring*

By field mapping, core drilling and laboratory testing parameters relevant for tunnel design can be estimated. This entails a number of uncertainties in mechanical (e.g. rock material, primary stress condition, hydromechanical condition) and spatial (e.g. location of lithology boundaries, joint orientation and distance, fault zone thickness) aspects. In this chapter focus is laid on field observations estimating joint orientation and distance.
Core drillings pertain to 1D measurements. One mentioned problem is the drilling direction in the field. By changing the orientation of drill core axis the sample may look completely different and some joint sets stay unidentified (Fig. 1). Just analysing the sample doesn’t lead to the joint orientation, hence additional borehole televiewing systems are necessary.

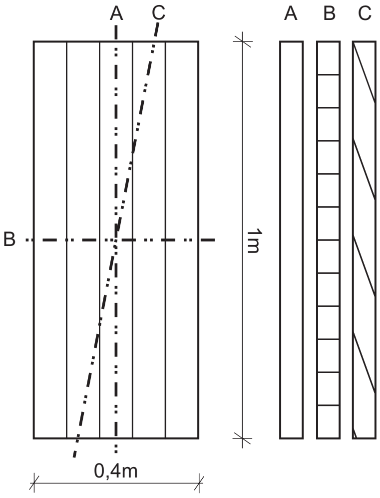

Figure 1. Samples of three different drilling directions

Field mapping on an outcrop pertains to 2D measurements and delivers more information about joint orientation and distance than e.g. a 1D core drill does, although even an outcrop analysis offers potential for misunderstandings.

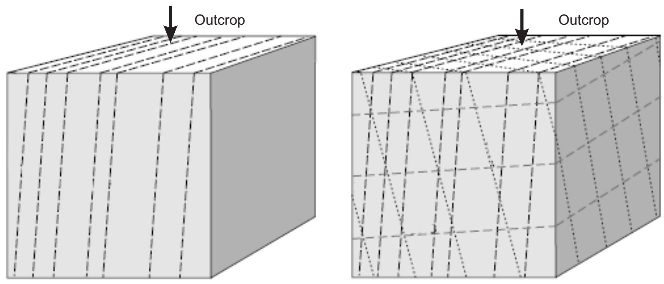

Figure 2. Joint sets and joint set spacing. (Palmström 2005)

Having a look at Figure 2 makes clear that estimation of a joint set striking parallelly and dipping similarly to the outcrop section is not feasible, as well as two joint sets with the selfsame dip direction are hardly distinguishable when just analysing a 2D section.
Palmström (2005) also remarks that for one joint set as in Fig. 2 (left) measuring the spacing is easy. But when more than one joint set exists as in Fig.2 (right) calculating the spacing is more complex.

2.3 *Difficulties with RQD*

The rock quality designation RQD is a measure for the core recovery and is meant to indicate the degree of fracturing. It gives the degree of core pieces which length is longer than 100mm in a section of the core drill. Here appears already the first problem.
Dividing the sample in only two parts, smaller and bigger than 10cm, causes a generalization which allows an output of one RQD for lots of completely different samples. Palmström (2001) already showed this phenomenon (Fig.3).

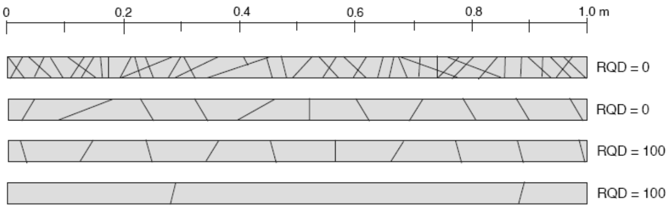

Figure 3. Examples of RQD values for various joint densities along drill cores (Palmström 2001).

The RQD only covers a limited part of the range of jointing as shown in Fig.4 which reduces the applicability of RQD in characterizing the jointing density and depicts the importance of its careful application.

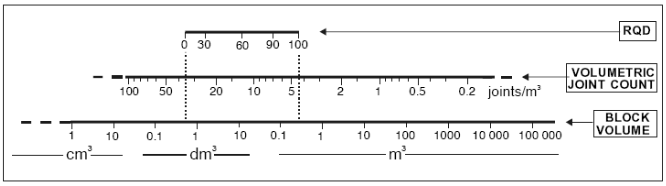

Figure 4. Block size (Vb) and volumetric joint count (Jv) cover a significantly larger interval of the jointing than the RQD (Palmström 2001).

The RQD is obtained by drill core samples, a procedure which contains an amount of uncertainties. First mentioned problem is the drilling direction. By changing the orientation of drill core axis the sample may look completely different as already described in chapter 2.2. Having a look at Fig.1 direction A and C represent a RQD of 100, direction B gives a RQD of 0.
Furthermore waviness or continuity of a joint cannot be established only by studying the drill core sample.
Another fact must be considered working with RQD. The drilling quality and core handling have an effect on the result as well.

2.4 *Difficulties with GSI*

"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 involves some subjectivity. Hence long-term experiences and sound judgment is required to successfully apply the GSI system." (Cai et al 2004) Therefore Cai et al devise a different approach, built on the concept of block volume and joint condition. The basic idea is adding measurable quantitative input for quantitative output in order to ensure a system less dependent on experience. Block size is now supplemented with block volume factor V_b and joint condition with the quantitative joint condition factor J_C similar to the one of RMi depending on large-scale waviness, small-scale smoothness and joint alteration factor.
Comparison of results estimated in the usual way with those after Cai et al may serve as an affirmation during underground structure design.

2.5 *Parameters not taken into account in classification systems*

2.5.1 *Tunnel radius*

As already shown in Tab.1 the tunnel radius is disregarded in all mentioned classification systems, although it can have a huge influence on the system behaviour. Comparing two models with same joint spacing but one radius is half of the other, a completely different displacement pattern can occur.
Two numerical calculations, one with radius of 3.0m, one of 1.5m and a joint set of 35° and 2.0m distance were executed. In case of smaller radius the joints have less impact on deformation than the material itself determining the behaviour. However in case of bigger radius joints control the deformation pattern. (Compare Fig. 5)
This simple example once again emphasises the importance of the ratio between the tunnel radius and the joint spacing and persistence.

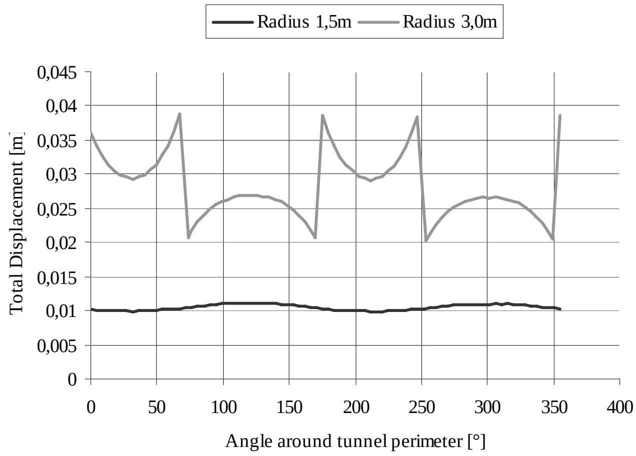

Figure 5. Total displacement patterns for calculations with 1.5m and 3.0m radius along the unwound cross section (beginning at left sidewall, anticlockwise).

3 NUMERICAL SIMULATION

3.1 *General*

By means of numerical simulations with FEM program Phase2 (Rockscience) and DEM program UDEC (Itasca 2004) five reference cases modelling a jointed rock with all classification systems (RMR, RMi, GSI) as well as with a discontinuous model were calculated in order to depict the drawbacks and simplification entailed by upscaling methods. Results scatter enormously in some cases as of course block kinematics is not considered in homogenous ground models, as suggested by RMR, RMi or GSI.
A numerical parametric study was accomplished, investigating two joint sets varying their distances from 1.5 m to 3.5 m (0.5m steps) with angles of +60° and -60° from horizontal in hard rock. The primary stress state is assumed to be homogenous with a K_0 value of 1.0 and an overburden of 600m. The radius is 3m, the water conditions are defined as dry. The rock mass and joint parameters for calculations can be taken from Table 2.

Table 2. Input parameters for calculations

Rock mass parameters		Joint parameters	
Density	27kN/m³	JKN	14000 Mpa
E	20 000 Mpa	JKS	5400 Mpa
ν	0.3	Cohesion	0 Mpa
K	17000 Mpa	Friction	15°
G	8000 Mpa	Filling	sand, silt
Cohesion	10 Mpa	Opening	5mm
Friction	38°	Surface	smooth, undulating

3.2 *Simplifications and Drawbacks*

3.2.1 *RMR*

As for the reference cases the RQD differs from 90 to 100% depending on the orientation and placement of the core drillings it doesn't make any difference in the rating for all 5 distances (from 1.5m to 3.5m in 0.5m steps) giving the value 15, which is somehow conflicting with actual knowledge about the joint sets.

The relatively course classification of the joint spacing in RMR leads to a jump from a rating of 20 for spacing larger than 2m to 15 for joint spacings between 0.6 and 2m. As the ratings of all other parameters remain the same, RMRs of 77 and 72 are obtained. Both fall into class II good rock for which a friction of 35-45° and a cohesion of 0.3-0.4 MPa is recommended. As the intact rock has a friction of 38°, 35° and 0.3 MPa cohesion for the rock mass is chosen.

The rock mass modulus was calculated with four differrent formulas invented by Bieniawski (1978), Mitri et al (1994), Read et al (1999) and Verman et al (1997) Three of them gave a rock mass modulus higher than that of the intact rock, leaving just one feasible modulus after Nicholson and Bieniawski, where E_i and RMR are input parameters.

The output are two calculations for all 5 cases with a homogenous ground as RMR presumes, a K_0 value of 1.0 determined for the study and E_{RM} = 7100 MPa (1.5, 2.0m) and 8600 MPa (2.5, 3.0, 3.5m). These calculations produce a total deformation of 8.1mm or 6.8mm along the intrados. (Fig.7)

Table 3. Parameters estimated with RMR

Spacing	RMR	c	φ	E_{rm}
[m]	[-]	[MPa]	[°]	[MPa]
1.5, 2.0	72	0,3	35	7100
2.5, 3.0, 3.5	77	0,3	35	8600

3.2.2 *RMi*

As J_C depends on joint roughness, joint alteration and joint size it remains the same for all 5 reference cases. The joint orientation has no influence at all, although it may have an impact on the failure mechanism itself. Nevertheless the block volume is an input parameter and of course increases with increasing joint distance, so far leading to 5 different RMi: 6.5, 8.7, 10.7, 12.8, 14.9 for distances of 1.5m, 2.0m, 2.5m, 3.0m, 3.5m and five different s, m_b and E_{RM}. The rock mass modulus was calculated with the formula after Palmström & Singh (2001).

RMi once again presumes a homogenous ground. Calculations with the RMi-parameter set and a K_0 value of 1.0 produce a homogenous deformation pattern around the tunnel intrados. Total deformations are 3.9mm, 3.5mm, 3.2mm, 3.0mm and 2.8mm for spacings of 1.5m, 2.0m, 2.5m, 3.0m, 3.5m. (Fig.7)

Table 4. Parameters estimated with RMi

Spacing	RMi	s	m_b	E_{rm}
[m]	[-]	[-]	[-]	[MPa]
1,5	6.5	0,025	8,63	14800
2,0	8.7	0,045	10,83	16600
2,5	10.7	0,069	11,88	18100
3,0	12.8	0,098	13,32	19400
3,5	14.9	0,132	14,66	20600

3.2.3 *GSI*

"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 involves some subjectivity. Hence long-term experiences and sound judgment is required to successfully apply the GSI system." (Cai et al 2004) Therefore Cai et al devise a different approach, built on the concept of block volume and joint condition. The basic idea is adding measurable quantitative input for quantitative output in order to ensure a system less dependent on experience. Block size is now supplemented with block volume factor V_b and joint condition with the quantitative joint condition factor J_C similar to the one used for RMi depending on large-scale waviness, small-scale smoothness and joint alteration factor.

Comparison of results estimated on the usual way with those after Cai et al may serve as an affirmation during underground structure design.

Nevertheless GSI allows calculating rock mass parameters s, m_b and E_{RM}.

For the reference cases five different GSI values estimated with the common chart (45, 50, 55, 60, 65 for 1.5m, 2.0m, 2,5m, 3.0m, 3.5m) which are allocated in "fair surface conditions" deduce 5 parameter sets for numerical calculations. The method after Cai et al (2004) delivers GSI values between 45 and 50 thus indicates a slight overestimation by common chart method for some of the reference cases. This endorses once again the untypical low values for such good rock material caused by the joint conditions. GSI values between 45 and 60 were determined as they are an approximation that corresponds to both methods.

The rock mass modulus was calculated with formulas after Hoek et al (2002) with the input parameters disturbance factor D, intact uniaxial compressive strength σ_{ci} and GSI and Hoek & Diederichs (2005) with intact modulus E_i and GSI. Both formulas give reasonable, similar results.

For GSI once again the basic idea is smearing the joints over the rock mass and herewith creating a homogenous ground model.

The numerical calculations produce a total deformation of 12.5mm, 9.9mm, 8.2mm, 6.6mm, 5.3mm for the five cases. (Fig.7)

Table 5. Parameters estimated with GSI

Spacing	GSI	s	m_b	E_{rm}
[m]	[-]	[-]	[-]	[MPa]
1,5	45	0,002	3,93	4640
2,0	49	0,004	4,53	5900
2,5	52	0,005	5,04	7050
3,0	56	0,008	5,82	8820
3,5	60	0,012	6,71	10900

3.2.4 *Comparison of results produced by RMR, RMi and GSI*

The displacement pattern is of course homogenous for all three models, as the K_0 value is set to 1.0 and the joints were smeared over the rock mass creating a homogenous ground. Worth mentioning the significance of such a model is questionable when actually dealing with block kinematics. The real displacement pattern may stay undetected.

Comparing the results of all three classification systems shows deformations from 13mm to 4mm which at least indicates sort of convergence within the systems.

RMi yields the smallest displacements of 3-4mm of all five reference cases.

RMR doesn't distinguish between all cases suggesting just one class (II good rock). However two cases remain due to two different E-moduli. The displacements range from 7 to 8mm.

Only calculations based on GSI deliver a bigger spread of resulting displacements between 13 and 5mm.

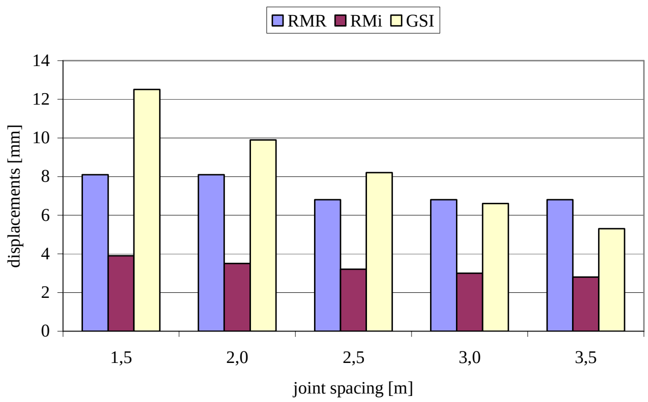

Figure 6. Comparison of displacements for RMR, RMi and GSI and the five reference joint distances.

3.3 *Discontinuous model*

Another approach to determine ground behaviour governed by joint sets is using a discontinuous model. Hereby the rock material is generated with mechanical, intact rock parameters, joints are created separately described by their own mechanical conditions (see Tab.2). This entails the possibility of describing the real failure mechanism as blocks are actually able to fall out.

This approach yields five different numerical calculations for the five spacings of the reference case giving reasonable results for each.

Fig.6 shows the different failure behaviours: After studying the results a limit for block falling and normal material deformation was set.

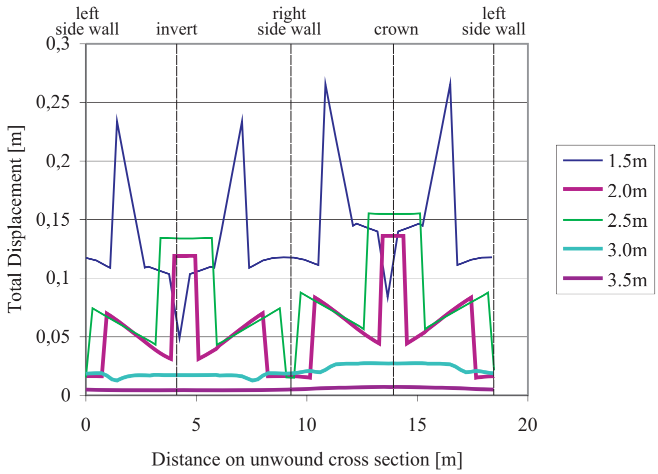

Figure 7. Displacements around unwound cross section for the five reference cases (beginning at left sidewall anticlockwise)

In case of smallest distance between joints the deformation is highest and blocks fall out at shoulders and sidewalls as it is also observable in the cross section in Fig.8. In cases with spacing of 2.0 and 2.5m the blocks at crown and invert are squeezed out. (Fig.9) The only difference between those two is the block size which is bigger in case of bigger distance between joints.

For cases with spacing of 3.0m and 3.5m the deformation pattern seems to be homogenous and displacements are between 10 and 30mm for 3.0m and 4 to 7mm for 3.5m spacing. This makes sense as the radius of the tunnel is at least same size as the distance of the joints, reducing the influence of the joints.

Basically those five reference cases show 3 different types of failure.

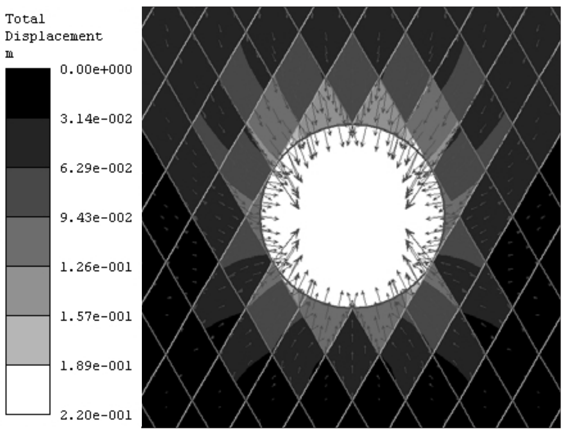

Figure 8. Displacements of cases with 1.5m joint spacing.

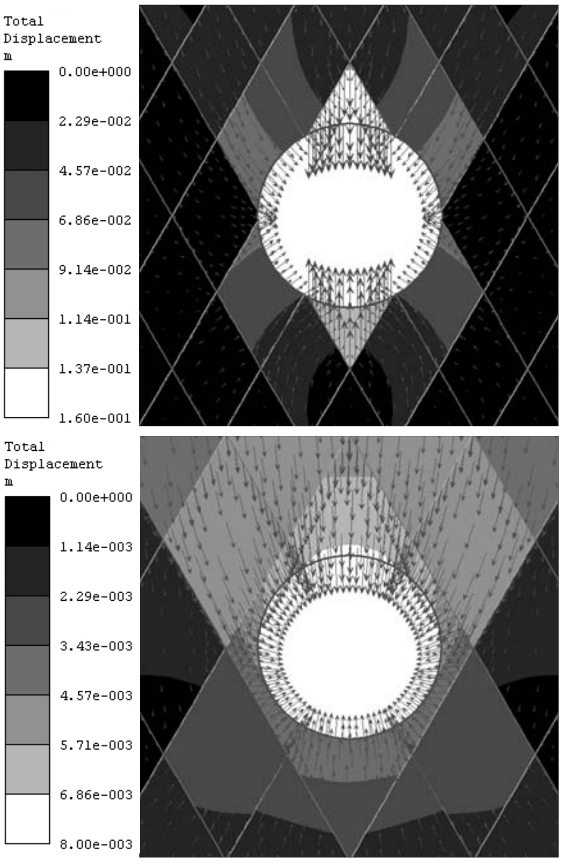

Figure 9. Displacements of cases with 2.5m joint spacing (above), 3.5m (below).

3.4 *Comparison of results*

Calculations based on rock mass parameters derived from classification systems yield homogenous deformations in mm scale. When using a discontinuous model, clearly the kinematical processes caused by the joints can be seen.
For the cases, where joint spacing is large (>3.0m) both approaches provide similar results. In case of joint spacings smaller than the tunnel radius, discontinuity controlled failure dominates the behaviour. Thus for such cases the use of a discontinuum model is strongly recommended.
The models presented here contain persistent joints. This definitely leads to “pessimistic” results. Further work will deal with realistic, non-continuous joint systems.

4 CONCLUSION

It is obvious that simplifications in the model can lead to the loss of important information in the output of an analysis. Costly modifications of the design during construction or even accidents can be the result of such a wrong assessment. This emphasises the importance of diligently selecting the methods for determining ground parameters and analysis methods for specific geotechnical conditions.
Design based on classification systems has to be accomplished carefully and deliberately, with keeping in mind the plethora of simplifications and drawbacks involved.
Comparing results of the discontinuous approach and classification systems approves the assumption that the idea of classification may hold true for some cases but as well may not for others. Designer must pay special attention on failure mechanisms, especially when dealing in jointed ground conditions where block falling is suspected.
The parametric study showed the tremendous difference in results and depicts the importance of an unbiased, hierarchical design approach.

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