

Distinct Element Modeling for the Stability Analysis of Fractured Rock Slopes: Case of Dolerite Slope from the Mamelle Hill, Dakar, Senegal

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ABSTRACT

The stability of fractured rock slopes is strongly influenced by the mechanical and kinematic behaviour of discontinuity networks. This study focuses on the analysis of the stability and failure mechanism of a rock slope composed of dolerite in the western Mamelle area of Dakar, using the distinct element method calculation code. The geomechanical characterization of the rock mass was carried out using the RMR and GSI classification systems, yielding values of 63 respectively, corresponding to a good quality rock mass. The numerical modelling results show that the mechanical response of the slope is primarily controlled by the network of discontinuities. The maximum calculated displacement is 9.574×10^{-4} m, while the maximum shear displacement along the joints reaches 6.366×10^{-4} m. The stress field remains predominantly compressive, with no significant concentration of tensile stresses. The results indicate that the potential instability mechanism is primarily associated with shearing and sliding along the discontinuities, rather than a deep or circular failure mechanism. Finally, the stability assessment yields a safety factor of 3.61, confirming the rock mass's very good overall stability.

Keywords: Dolerite; Slope stability; Distinct Element Method; Discontinuity network; Factor of safety; Mamelle Hill.

INTRODUCTION

The stability of rock masses is a major geotechnical issue due to the risks associated with rock mass movements. Unlike a continuous and homogeneous material, rock masses generally consist of an intact rock matrix intersected by net network of discontinuities such as joints, fractures, cracks, faults, and bedding planes. The mechanical behavior of these structures is influenced not only by the intrinsic properties of the rock matrix but also by those of the discontinuities, which divide the mass into distinct blocks prone to sliding, rotation, and tipping. Consequently, conventional based approaches may have limitations in adequately the blocky nature and discontinuity-controlled behaviour of fractured rock masses.

The Mamelle hills in Dakar (Senegal) are composed predominantly of volcanic formations, including dolerites, basanites and tuffs [1]. These formations exhibit a complex geological structure characterized by significant fracturing and variable degrees of weathering. The geotechnical characterization and stability modeling of the rock slopes at this area are critical to

preventing slope instability risks in this rapidly urbanizing coastal area.

This study aims to characterize the geomechanical conditions of a dolerite rock slope located in the western Mamelles area of Dakar using the Rock Mass Rating (RMR) system proposed by Bieniawski [2] and the Geological Strength Index (GSI) [3]. The resulting geomechanical characterization provides the input parameters required for numerical modelling using the two-dimensional Universal Distinct Element Code (UDEC 2D). Unlike continuum-based approaches, the distinct element method explicitly represents the rock mass as an assemblage of blocks separated by discontinuities [4], allowing the analysis of block displacements, joint deformation, and potential failure kinematics. The numerical analysis is used to investigate the stress and displacement fields, assess the mobilization of discontinuities, identify the potential instability mechanism, and determine the factor of safety of the studied rock slope.

MATERIALS & METHODS

Geographical and Geological Study

The study area is situated in the Dakar region, at the western tip of the Cap-Vert Peninsula, more specifically in the Mamelles massif. The Cap-Vert Peninsula forms the

westernmost part of the Senegal-Mauritanian coastal basin, which lies on the western edge of the West African craton. Its geographical position and geological context make this region of particular interest for the study of volcanic formations and their geomechanical behaviour.

The geological history of the study area fits within the broader context of the Cap-Vert Peninsula, which was characterized by significant volcanic activity during the Tertiary and Quaternary periods (Figure 1). Evidence of Tertiary volcanism is widely dispersed across the Cap-Vert Peninsula in Dakar as well as in the Thies region, whereas Quaternary volcanic formations are mainly concentrated in the western part of the peninsula [5, 6]. The Mamelles volcanic system, whose eruptive center is located in the west of the peninsula, produced pyroclastic deposits (cinerites, tuffs, pumice, volcanic bombs) and doleritic basanite flows deposited between the lower Pleistocene (1.4 Ma) and the middle Pleistocene [7]. These formations are widely exposed along the northwestern coast of the peninsula, from Pointe de Fann to Yoff, where they form coastal cliffs as well as the Mamelles Hills. The sequence and arrangement of these formations, as well as their fracturing, play a decisive role in the structuring of the rock mass under study.

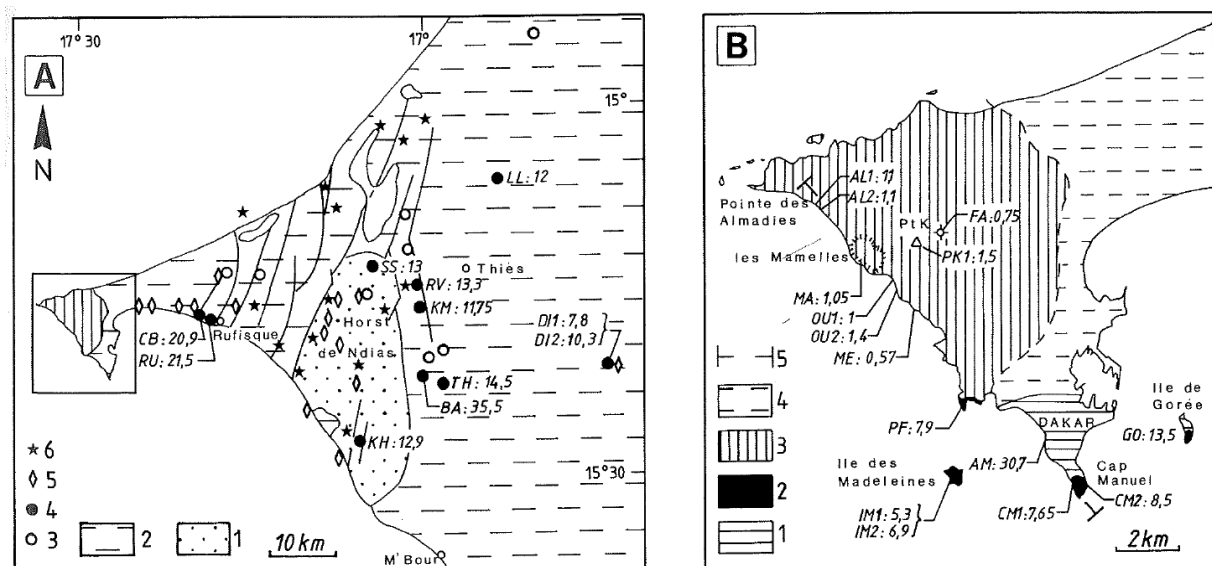


Figure 1. Outcrops of Cenozoic volcanic rocks in Cap-Vert peninsula [6]: (a) Cap-Vert peninsula and Thiès areas; (b) The Cenozoic volcanism of the Head of the Cap-Vert peninsula.

The studied dolerite is characterized by a fine to medium-grained doleritic texture. It is composed predominantly of plagioclase and

pyroxene, with subordinate olivine and magnetite, and minor amounts of quartz and feldspathoids (Figure 2).



Figure 2. Outcrop of the studied doleritic rock mass at the Mamelles site

Rock Mass Classification

In our study, the rock mass was characterized using the Rock Mass Rating (*RMR*) proposed by Bieniawski and the Geological Strength Index (*GSI*) developed by Hoek and Brown. This classification aims to assess the quality of the rock mass and to determine the geomechanical parameters required for its analysis and modelling. Measurements were taken at several monitoring stations on the doleritic facies representative of the rock mass under study. For each station, the parameters required for the *RMR* assessment were determined on the basis of field observations and structural measurements. These include, in particular: strength of intact rock (R_1); *RQD* (R_2); spacing of discontinuities (R_4); condition of discontinuities (R_5) and groundwater condition (R_6). The orientation, number of joint families and their spacing were determined from measurements taken along a survey line crossing the monitoring station. These observations made it possible to characterise the structure of the massif and to identify the main families of joints likely to

control its mechanical behaviour. In the absence of rock cores, the *RQD* was estimated indirectly from the joint frequency, using the relationship proposed by Priest and Hudson [8].

$$RQD = 100 \times (0,1\lambda + 1) \times e^{-0,1\lambda} \quad (1)$$

Where λ is the joint frequency ($\lambda = 1/s_j$).

The value of the *RMR* varies from 0 to 100. The sum of the six parameters is used to estimate the *RMR* value.

$$RMR = R_1 + R_2 + R_3 + R_4 + R_5 + R_6 \quad (2)$$

The Geological Strength Index (*GSI*) is introduced by Hoek (1994), Hoek et al. (1995) and improved by Hoek and Brown (1997). It was introduced specifically for estimating rock properties. Relations have been proposed by Hoek and Brown (1997) to estimate the *GSI* [9] from the *RMR*.

$$GSI = RMR' - 5 \quad (3) \quad \text{if } RMR' > 23$$

With *RMR'*, the value of Bieniawski's *RMR*₈₉ has been adjusted [10] to take into account a score of 15 for hydraulic

conditions and a score of zero for joint orientation.



Figure 3. In situ Schmidt hammer rebound testing for the geomechanical characterization

Numerical Modelling

The numerical modelling of the slope was carried out using the UDEC 2D version 7 code (Universal Distinct Element Code, Itasca) [11], based on Cundall's distinct element method [12]. This method represents the fractured rock mass as an assembly of rigid or deformable blocks interacting via joints

Basic Principle of Distinct Element Method

The distinct element method is based on a dynamic algorithm that solves the equations of motion for the system of blocks using an explicit finite difference method [13]. In the distinct element method, the laws of motion and the behavioural relationships of the joints are applied at each time step. Integration of the law of motion yields the new positions of the blocks and, consequently, the displacement increments at the joints. A force-displacement relationship describing the behaviour of the joint at the points of contact is then used to obtain the forces acting on the blocks at the next time step. The

equation of motion for rigid blocks is determined by the magnitude and direction of the resultant of the forces and moments acting at the centre of gravity of these blocks. Newton's second law of motion is expressed as:

$$\frac{\partial \dot{u}}{\partial t} = \frac{F(t)}{m} \quad (4)$$

For blocks in two dimensions that are acted upon by several forces as well as gravity, the velocity equations become:

$$\dot{u}_i^{(t+\Delta t/2)} = \dot{u}_i^{(t-\Delta t/2)} + \left(\frac{F_i^t}{m} + g_i \right) \Delta t \quad (5)$$

Where

$\dot{u}$ velocity components of block

F resultant of the contact forces and external forces acting on the blocks

m mass of the blocks

g_i components of gravitational acceleration

The calculation cycles for rigid and deformable blocks are shown in Figure 4.

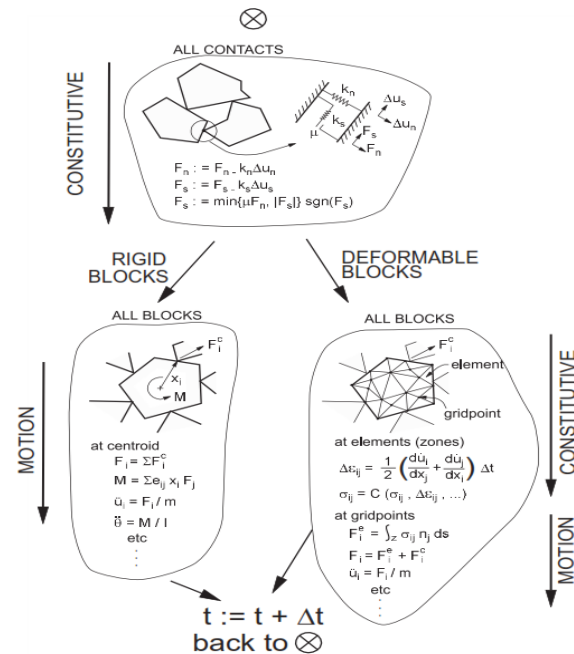


Figure 4. Calculation cycle used in distinct element method [11]

Model Configuration and Boundary Conditions

Figure 5 shows the geometry of the rock slope modelled in UDEC 7.0. The model represents a rock slope approximately 10 m high, with a gradient of around 55°. The rock mass is modelled as a discontinuous medium separated by two main families of joints. These joints have orientations of 60° and – 70° respectively, with average spacings of 0.20 m and 0.15 m respectively. The

dimensions of the computational domain were chosen to minimise the influence of the boundaries on the behaviour of the rock mass in the vicinity of the slope. The boundary conditions replicate horizontal and vertical displacements that are completely restrained at the base of the model, whilst only the horizontal displacement is restrained at the lateral boundaries, with the vertical displacement remaining free there.

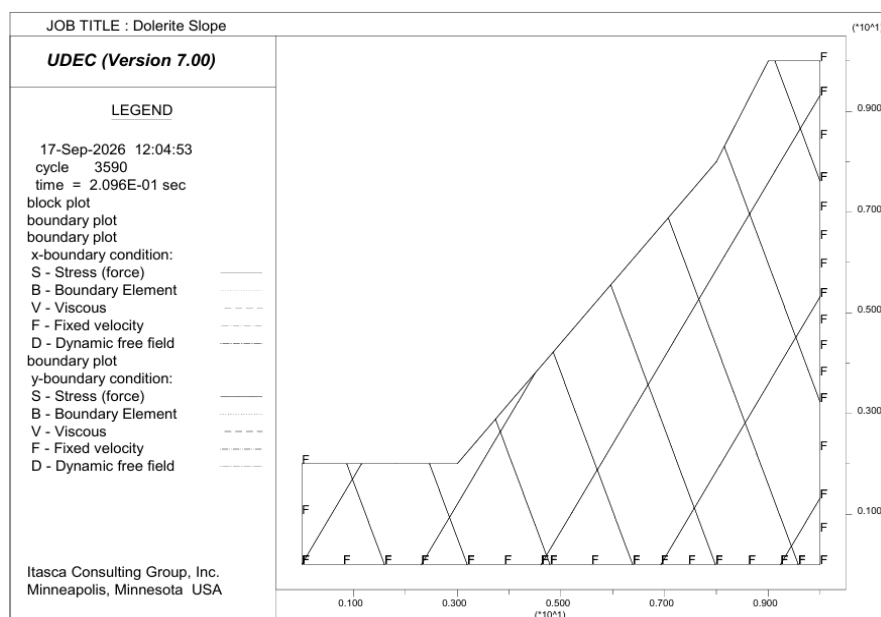


Figure 5. Geometrie and boundary conditions of the slope model

Model Constitutive and Parameters

As the rock mass consists of the rock matrix and joints, two behavioural models were used: the Mohr–Coulomb model and the Coulomb slip model, to describe the mechanical behaviour of the intact rock and the joints respectively. The mathematical formulation of the Mohr–Coulomb failure criterion in the principal stress plane is given by:

$$f = \sigma_1 - \sigma_3 N_\varphi + 2c\sqrt{N_\varphi} \quad (6)$$

$$N_\varphi = (1 + \sin\varphi)/(1 - \sin\varphi) \quad (7)$$

σ_1 and σ_2 are the principal stresses

The equivalent Mohr–Coulomb parameters (c' and φ') were determined using the method described by Hoek, Carranza-Torres and Corkum (2002) [14]. The modulus of deformation of the massif was estimated using the generalised equation of Hoek, Carranza-Torres and Corkum (2002).

$$\varphi' = \sin^{-1} \left[\frac{6am_b(s+m_b\sigma'_{3a})^{a-1}}{2(1+a)(2+a)+6am_b(s+m_b\sigma'_{3a})^{a-1}} \right] \quad (8)$$

$$c' = \frac{\sigma_{ci}[(1+2a)s+(1-a)m_b\sigma'_{3a}](s+m_b\sigma'_{3a})^{a-1}}{(1+a)(2+a)\sqrt{1+(6am_b(s+m_b\sigma'_{3a})^{a-1})/((1+a)(2+a))}} \quad (9)$$

$$E_m(\text{GPa}) = \left(1 - \frac{D}{2}\right) \sqrt{\frac{\sigma_{ci}}{100}} \cdot 10^{\left(\frac{\sigma_{3n} = \sigma'_{3mx} / \sigma_{ci}}{(GSI-10)}\right)} \quad (10)$$

D is the disturbance factor for the rock mass (D = 0, undisturbed rock mass)

RESULTS

Rock Mass Classification and Material Properties

The rock mass forming our slope has a simple compressive strength of 98 MPa. The RQD, estimated using Equation 1, gives a value of 73.3 per cent. The joints running through the slope in both directions are slightly weathered and rough, with spacings ranging from 0.2 m to 0.15 m. These joints have polished surfaces filled with material resulting from rock weathering. The rock

mass is characterised by damp conditions, due to its proximity to the ocean and the permanent presence of water at the foot of the slope. All the parameters considered have resulted in an RMR value of 63, corresponding to Class II, which is characteristic of a good-quality rock mass according to Bieniawski's classification. The collected data were processed in accordance with the RMR classification system and presented in Table 1.

Table 1. RMR values of rock masses

Parametres	Values	Rating
Uniaxial compression strength (MPa)	97.00	7
RQD (%)	73.30	13
Discontinuity spacing (mm)	60-200	8
Discontinuity conditions: aperture; roughness; filling; weathering	Slightly rough surfaces separation < 1 mm slightly weathered walls	25
Groundwater condition	wet	10
RMR	-	63

The GSI, determined using equation (2), is equal to 63. This value also indicates a rock mass of relatively good quality, although it exhibits heterogeneities associated with fracturing and the weathering of the discontinuities.

The mechanical properties selected for the intact rock and the discontinuities, and used in the numerical modelling, are presented in Tables 2 and 3.

Numerical Stability Analysis Displacement, Stress and Joint Kinematic

The results show that the mechanical response of the rock slope is strongly governed by the network of discontinuities. The maximum displacement reaches 9.574×10^{-4} m (Figure 6.a) and is concentrated mainly along the slope, increasing towards the crest (Figure 6.b). The stress field shows a predominantly compressive stress state; with a minimum principal stress of the order of -6.15×10^5 Pa and a maximum principal stress that is practically zero (Figure 6.c). The gradual change in the orientation of the

principal stresses towards the flank reflects a redistribution of stresses induced by gravity, the geometry of the slope and interactions between blocks (Figure 6.d). The behaviour of the interfaces shows that deformations are localised along the discontinuities, with a maximum shear displacement of 6.366×10^{-4} m (Figure 6.e). At the critical state, the velocity vectors are mainly directed outwards and downwards, indicating a progressive movement of the rock blocks towards the free surface.

Table 2. Mechanical properties of the material on the slope

Rock mass property	Values
Density (kg/m ³)	2600
Bulk modulus K (GPa)	17.435
Shear modulus G (GPa)	8.047
Friction angle ϕ (°)	65.38
Cohesion c (kPa)	1980
Poisson ratio (ν)	0.3

Table 3. Mechanical properties of the discontinuities on the slope

Joint Property	Values
Normal stiffness K_n (MPa/mm)	10.75
Shear stiffness K_s (MPa/mm)	1.50
Friction angle ϕ_j (°)	23
Cohesion c_j (kPa)	20
Dip (°)	60 ; 260
Spacing (m)	0.2 ; 0.15

Critical State and Factor of Safety

The strength reduction analysis yields a factor of safety of **$FS = 3.61$** . The resulting critical state is characterized by an increase in joint shear displacement and a coherent field of block velocities directed downward and outward, reflecting a progressive localization of deformation along the discontinuity network.

Failure Mechanism

The numerical results as a whole demonstrate that the potential instability is essentially governed by shear along rock joints and the displacement of blocks towards the void. The absence of tensile stresses and the localisation of deformations at the discontinuities confirm a failure mechanism controlled by the structure of the rock mass.

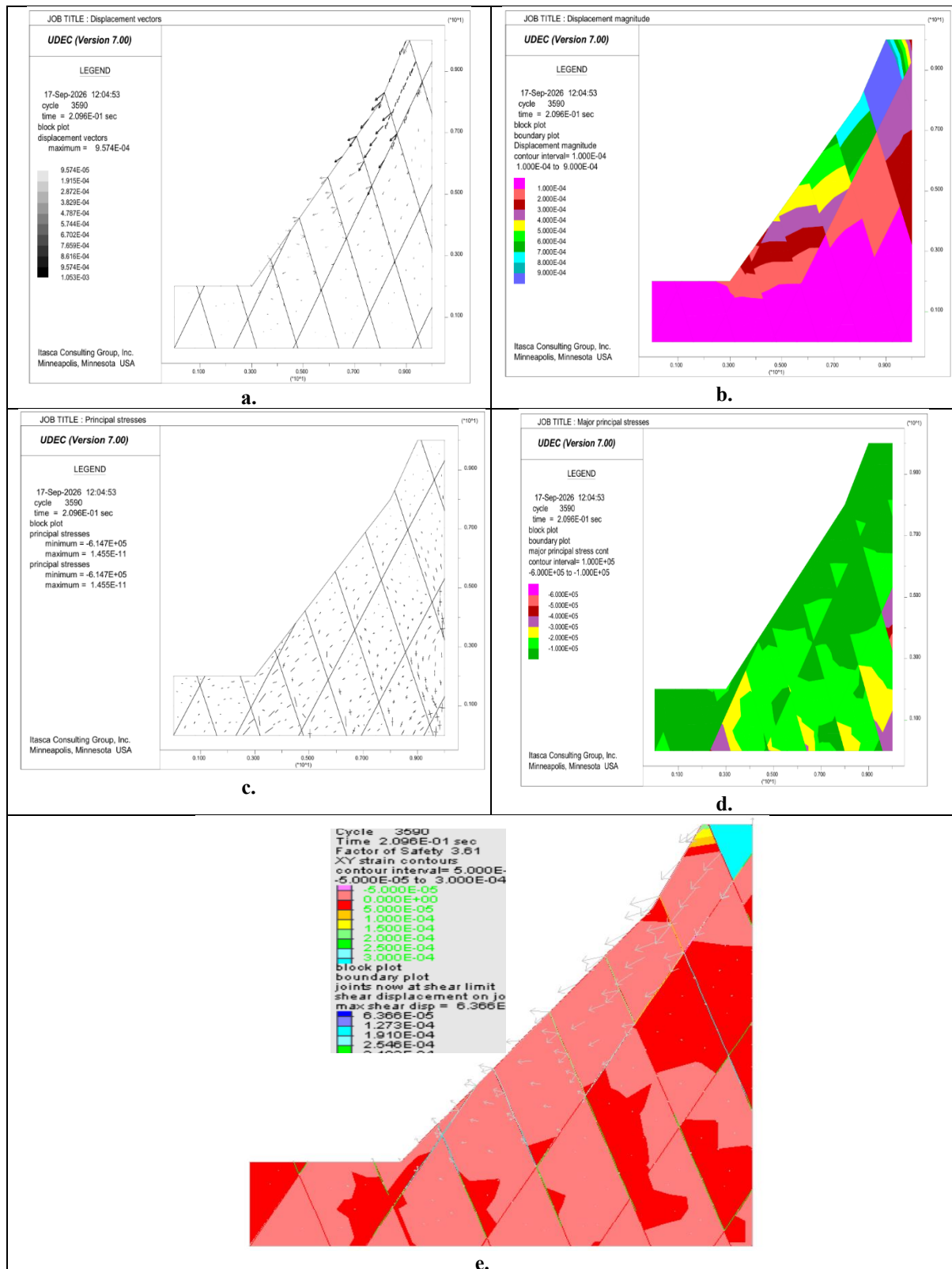


Figure 6: UDEC results: (a) displacement vectors; (b) displacement magnitude; (c) Principal stress vectors; (d) contours of the major principal stress; (e) shear displacement along joints and velocity vectors at the critical state.

DISCUSSION

The geomechanical characterization of the dolerite rock mass reveals a good quality,

defined by *RMR* and *GSI* values of 63. Numerical simulations conducted using UDEC 2D demonstrate that the structural

response of the slope is strictly governed by the discontinuity network. The maximum displacements, which remain very low in magnitude, are concentrated along the slope face and exhibit an increasing gradient toward the crest. The stress field remains predominantly compressive throughout the profile. The failure kinematics consists of preferential sliding along joint interfaces toward the free space. Stability analysis using the shear strength reduction method yields a high factor of safety ($FS=3.61$). This level of safety confirms that significant degradation of the joint's shear properties is required to initiate a limit state of failure. These results validate an instability mechanism primarily controlled by the structural behavior of discontinuities, ruling out any risk of deep-seated or circular failure within the rock mass.

CONCLUSION

This study assessed the mechanical behaviour and stability of a dolerite rock slope in the western Mamelle area of Dakar, by combining a geomechanical characterisation of the rock mass with numerical modelling using the distinct element method and the UDEC 2D code. The geomechanical classification indicated a rock mass of good quality, but one exhibiting a network of discontinuities likely to locally control its mechanical behaviour. The modelling results show that the slope's response is primarily governed by the interaction between the rock blocks and the discontinuities. The stress distribution remains predominantly compressive, whilst the localisation of deformations at the interfaces confirms the predominant role of the discontinuities in the massif's response. The analysis based on shear strength reduction yields a safety factor of 3.61, indicating that the slope has a margin of stability. However, the results also show that changes in stability are likely to be strongly influenced by the strength properties of the joints. The use of the UDEC 2D calculation code provides a better understanding of the discontinuous behaviour of the rock mass

and highlights a mechanism of instability controlled by shear and slippage along the discontinuities, rather than by a deep or circular failure.

To complement this analysis, a sensitivity analysis focusing in particular on the joint strength parameters and hydraulic conditions would make it possible to better quantify their influence on the safety factor and on the progression of the instability mechanism.

Declaration by Authors

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