A DEM-BASED FACTOR TO DESIGN ROCK-SOCKETED PILES

CONSIDERING SOCKET ROUGHNESS

Gutiérrez-Ch J.G., Senent S., Melentijevic S. and Jimenez R.

ABSTRACT

The Distinct Element Method (DEM) has gained recent attention to study geotechnical designs with rock-concrete or rock-rock interfaces, such as rock-socketed piles. In this work, 3D DEM models with non-standard contacts laws (the Smooth-Joint and Flat-Joint contact models) are proposed to analyze the response of axially loaded rock-socketed piles with different sockets roughness, since socket roughness is a key factor affecting their side shear resistance that is not usually considered for pile design. DEM models are calibrated using experimental data, and the consequences of applying 2D models for calibration, to be subsequently used in a 3D analysis, are studied. Numerical results suggest that such DEM models can be employed to reproduce key aspects of the behavior of rock-socketed piles, such as their load and global stiffness-settlement response, their side shear resistance, and the damage at the rock-pile interface. Finally, an empirical factor $\alpha_{RF,1\%D}$ is proposed to estimate the side shear resistance of rock-socketed piles considering the socket roughness and the uniaxial compressive strength (UCS) of the weaker material (rock or pile) at the interface.

Keywords DEM, PFC (Particle Flow Code), Smooth Joint Contact Model, Flat Joint Contact Model, rock-socketed pile, socket roughness.
1. Introduction

Rock-socketed piles are a frequent type of deep foundation employed in many civil and architectural projects like bridge and harbor construction (Xing et al. 2014; Zhou et al. 2020), or multi-storey office and residential buildings (Karandikar 2018). Usually, the design of rock-socketed piles is based on the assumption that their working load is transferred to the surrounding rock by a combination of side shear resistance and of base resistance. But the side shear resistance is mobilized first for small deformations (or lower pile settlements) (Zhang 2004), so that understanding its behavior and load-transfer mechanisms is a key aspect for pile designers.

In the last decades, the side shear resistance of rock-socketed piles has been analyzed through laboratory or in-situ tests, such as (i) direct shear tests conducted on rock-concrete interfaces under a constant normal stiffness condition (Kodikara and Johnston 1994; Gu et al. 2013), (ii) small-scale pile load tests conducted at 1-g (Dai et al. 2017; Xu et al. 2020) or at higher g-levels in centrifuge facilities (Leung and Ko 1993; Dykeman and Valsangkar 1996), and (iii) in-situ full-load tests (Williams and Pells 1981; Ng et al. 2001; Seol and Jeong 2007). As a result, many empirical formulations have been proposed to estimate the side shear resistance of rock-socketed piles.

Despite recent efforts to consider the effects of (triangular) asperities on the side shear resistance of rock sockets (see e.g., Zhao et al. 2019; Zhou et al. 2020), most of the formulations proposed in the literature are still generalized as an exponential equation (Rezazadeh and Eslani 2017):
\[ \tau_{ave,peak} = \alpha \sigma_c^\beta \]  

where \( \tau_{ave,peak} \) is the average ultimate side shear resistance, \( \sigma_c \) is the intact uniaxial compressive strength (UCS) of the weaker material (pile or rock) at the socket, and \( \alpha \) and \( \beta \) are empirical factors that mainly depend on the type of rock. But Eq. (1) neglects the influence of other important aspects affecting the side shear resistance behavior, such as the rock mass rating (Melentijevic and Olalla 2014), the drilling method and drilling tools used (O’Neill et al. 1996; Nam and Vipulanandam 2008), or the roughness at the rock-pile interface, which has been shown to be one key factor affecting the response of rock-socketed piles (Pell et al. 1980; Horvath et al. 1983, Seidel and Collingwood 2001). Due to difficulties and costs associated to conducting laboratory and in-situ tests, numerical models –such as the Distinct Element Method (DEM), the Finite Element Method (FEM) or the Finite Difference Method (FDM)– have become an alternative to study the side shear resistance of rock-socketed piles (Melentijevic and Olalla, 2014; Gutiérrez-Ch and Melentijevic 2016; Gutiérrez-Ch et al. 2019) or the shear stress of rock-concrete interfaces (Tian et al. 2015; Gutiérrez-Ch et al. 2018; Saadat and Taheri 2020a). In particular, the DEM has been shown to adequately reproduce some key aspects of rough rock-socketed piles, such as their load-settlement response and the damage at the rock-concrete interface (Gutiérrez-Ch et al. 2020; Gutiérrez-Ch 2020). This opens the way to use DEM results, in combination with real data, to improve our current design practice which, as mentioned above, does not typically consider socket roughness. In that sense, this paper expands previous research by (i) analyzing the applicability of DEM to model
rock-socketed piles, providing additional information regarding the use of 2D models to calibrate the micro-mechanical parameters of DEM models, so that results can be employed for 3D socket analyses, and (ii) proposes new simple guidelines to preliminarily predict the average side shear resistance considering the socket roughness and the uniaxial compressive strength of the intact weaker material (rock or pile).

2. Calibration of FJCM and SJCM: comparison of 2D vs 3D models

3D numerical models built in the Particle Flow Code (PFC) (Itasca Consulting Group Inc. 2014) are used in this paper to model rock-socketed pile behaviour. PFC can simulate concrete or rock using a Bonded-Particle Model, which represents their microstructural features and larger-scale “interfaces” to simulate the macroscopic behaviour of intact materials (Potyondy 2015). In this work, the Flat-Joint Contact Model (FJCM) proposed by Potyondy (2012) and the Smooth-Joint Contact Model (SJCM) proposed by Pierce et al. (2007) are employed to model, respectively, the pile (concrete C1 and C2) and the rock bodies (sandstones S2 and S3, and gneiss G) and the pile-rock interface. Rock and concrete properties are listed in Table 1. The FJCM is employed due to its moment-resisting ability after bond breakage, providing a frictional finite-length interface after the bond is fully broken; thus, it tends to provide more appropriate compressive-to-tensile strength ratios for rocks (Potyondy 2018). On the other hand, the SJCM is employed due to its capabilities to reproduce the shear behaviour of rock-rock (Saadat and Taheri 2020b) or rock-concrete (Gutiérrez-Ch et al. 2018) interfaces more realistically, since particles at the joint plane can slide
past one another without over-riding one another (Pierce et al. 2007), avoiding inappropriate distribution of shear and normal forces at the joint plane.

[Table 1 approx. here]

One advantage of the proposed DEM approach is that the parameters of the contact models employed can be calibrated by comparison of 2D numerical simulation results with results of simple laboratory tests —e.g., UCS test, triaxial test, Brazilian test, direct shear (DS) test, etc.— (Potyondy 2015; Saadat and Taheri 2020b; Bahaaddini 2017). In this work, the micromechanical parameters of the FJCM and of SJCM are calibrated through an iterative process using, respectively, numerical simulations of UCS and DS tests, whose results are compared against experimental tests results reported by Gu et al. (2003) and Gutiérrez (2013). This calibration process has already been conducted by the authors in a previous work (see Gutiérrez-Ch et al. 2018, 2020 for details) and, the results of the micromechanical properties for the FJCM and for the SJCM are listed in Tables 2 and 3, respectively.

[Table 2 approx. here]

[Table 3 approx. here]

Note, however, that due to computational efficiency such calibration was conducted using 2D numerical samples: 2D models are much faster, and many model-runs are required in the calibration process (Gutiérrez-Ch et al. 2020). Since this could introduce uncertainties associated to the calibration process (results of a 3D model using parameters calibrated with a 2D model could be inadequate), a
more in-depth analysis described below is conducted to verify that this approach is valid for practice (see also the rock socket simulation results, which further address this issue in Section 3.2).

2.1. Effect of a 2D calibration of FJCM parameters

To validate the 2D calibration approach, Sandstone (S3) specimens with dimensions of 115.1 mm × 55 mm are employed; with micromechanical FJCM parameters reported by Gutiérrez-Ch et al. (2018) (see Table 2). To that end, once a 2D DEM model has been calibrated using UCS tests, another 3D DEM model is built using the same micromechanical parameters that were calibrated from the 2D DEM, so that differences between their simulated macromechanical properties – the UCS, $\sigma_c$, the Young’s modulus, $E$, and the Poisson’s ratio, $\nu$ – can be obtained.

For both 2D and 3D DEM UCS tests, the loading velocity was set to 0.1 m/s. This value was selected because, based on the sensitivity analyses conducted (not reported herein for brevity), it was found to be low enough to ensure that DEM specimens remain in quasi-static equilibrium, so that their macromechanical properties are not affected when the loading velocity is reduced (Potyondy and Cundall 2004).

Fig. 1a shows an example of the specimens generated in 2D and 3D, which employ 1,145 (for the 2D DEM) and 23,036 (3D) particles, respectively. Results are presented in Fig. 1b-c and Table 4. (Note that number of tests, $n$, and statistics of DEM results are reported, since several simulations have been carried out to reduce the influence of the initial seed value of the random-number generator used during genesis of the DEM specimen). As it can be observed, 2D and 3D samples
have a similar failure mechanism, and very similar macromechanical response \((\sigma_c, E, \text{ and } \nu)\), hence suggesting that micromechanical parameters calibrated in 2D can be employed for 3D simulations. There is a slight overestimation of the peak uniaxial compressive strength \((\sigma_{c,PFC3D}/\sigma_{c,PFC2D} = 1.1)\) and, in agreement with previous research (Ding et al. 2014), the coefficient of variations (COV) of results are lower when 3D DEM simulations are used.

Since results are very similar, one major reason to prefer a 2D calibration is computational time. Table 5 compares the time needed to run one UCS test in 2D or in 3D: note that the computational time is increased by 80 when a 3D model is used instead of a 2D model. This is a very significant difference, since many runs are needed during the calibration process; remember that it is a “trial and error” calibration.

2.2. Effect of a 2D calibration of SJCM parameters

To evaluate the differences due to a 2D calibration of SJCM parameters and its applicability to 3D simulations, 2D and 3D DS numerical tests on unbonded concrete-sandstone (S3) planar interfaces under CNL boundary conditions with different normal stresses \((\sigma_n)\) are conducted (see Fig. 2), using the micro-parameters reported by Gutiérrez-Ch et al. (2018) (see Tables 2 and 3). To simulate the DS tests, the displacements of the lower block are restraint, and a
horizontal velocity of 0.01 m/s is applied to the upper block from left to right. The shear displacements are obtained through the horizontal displacements recorded at the upper block, and the shear stress is obtained dividing the average force reaction on the upper left and lower right walls by the joint length (in 2D models) or by the joint area (in 3D models). Note that, for simplicity and efficiency, (i) 3D DEM simulations are conducted until a limit shear displacement of 1 mm, and (ii) only a relatively small joint is used. Also, to evaluate the advantage of using periodic boundary conditions (PBC) (Itasca Consulting Group Inc. 2014) when a 3D calibration is needed, additional DS test are conducted with 3D DEM using a thin slice of 10 mm and a PBC (see Fig. 2).

Results are presented in Fig. 3, showing that there is not a significant difference on the macro-response obtained from simulations conducted with the 2D and 3D DEM numerical models, even when such 3D models employ the micromechanical parameters obtained from 2D simulations: note that the peak shear strengths in both models are similar to the laboratory ones, and that the shear stiffnesses in 3D models is slightly higher than in 2D models. Also, results in Fig. 3 show that the response of 3D DEM models constructed using periodic boundary conditions are similar to those obtained using 2D or 3D simulations, but with a considerable reduction in calculation time (see Table 6). Again, results are similar and the main difference is computational time: 3D models are probably too expensive to be able to conduct a full calibration with them, so that 2D models are probably preferable to calibrate the SJCM parameters (see Table 6).
Finally, one last aspect regarding the applicability of 2D calibration of SJCM parameters to 3D simulations has to be analyzed: the particle motions at the rock-concrete interface. To further illustrate that behavior at the interface is mainly 2D, tracking balls were located at the rock-concrete interface of the DS test under $\sigma_n = 400$ kPa with PBC that was reported in Fig. 3, so that they can help evaluate the motions at such interface (see Fig. 4a; note that, to facilitate the illustration of tracking balls, concrete particles are not shown). Results are shown in Fig. 4b, and they suggest that the motions at the rock-concrete interface, except in the shear direction, are not substantial, hence supporting the conclusion that it is appropriate to employ 2D numerical models to simulate DS tests, and that if it were necessary to calibrate the SJCM parameters with 3D models, a good option would be to perform them with PBC.

3. Applicability of 3D DEM to model rock-socketed piles considering socket roughness

3.1. Rock-socketed pile numerical model

To evaluate the applicability of DEM to simulate axially loaded rock-socketed piles considering the socket roughness, 3D DEM simulations using PFC and the methodology developed by Gutiérrez-Ch et al. (2019, 2020) are used. A rock-socketed pile, and the corresponding 3D DEM model developed to represent it, are
presented in Fig. 5a-b. (The process to generate the numerical model is described in Gutiérrez-Ch et al. (2019, 2020) and it is not repeated here for the sake of brevity).

The nominal socket radius ($R$) and the nominal socket length ($L$) are 0.4 m and 0.8 m, respectively. To reduce the computational time, (i) a 45-degree angle portion of the pile (instead of the whole pile) is considered, (ii) pressure loads are applied on the wall head rock ($Q_2 = 0.1$ MPa) and on the wall head pile ($Q_1 = 0.125$ MPa) to simulate, respectively, the overlying soil stratum and the self-weight of the pile embedded in it, and (iii) the rock body is discretized into three zones (Fig. 5b) with increasing particle size, so that the smallest particles are located at the pile-rock interface. The minimum and maximum particle radii (i.e., $R_{\text{min}}$ and $R_{\text{max}}$) are defined based on a sensitivity analysis that identifies the particle size beyond which pile behavior becomes independent of particle size; in this case, it was found that this occurs when the minimum radius of particles is lower than 1 cm (i.e., $R_{\text{min}} \leq 1$ cm, see Fig. 6).

The base of the pile is made void, since only side shear resistance is considered herein (see Fig. 5(a)-(b)), and the FJCM and SJCM models are employed to simulate the pile and the rock bodies and the pile-rock interface, with the same micromechanical parameters that were calibrated by Gutiérrez-Ch et al. (2018) using real test results reported by Gu et al. (2003) and by Gutiérrez (2013) (see Gutiérrez-Ch et al. 2018 for details). As mentioned in Section 2, such micromechanical properties were calibrated using 2D models.
According to O’Neill et al. (1996), the typical shape recorded in sockets drilled with an auger tool in soft rock has a sinusoidal pattern. To simulate that, and to obtain different degrees of socket roughness, sinusoidal surfaces with asperity amplitudes of 4, 10, 20, 32.5 and 40 mm, and with a wavelength of 250 mm in all models, are employed (see Fig. 5c). Using the roughness factor (RF) of Horvath et al. (1983), defined as \( RF = (h_m \cdot L_t)/(R \cdot L) \) (where \( h_m \) is the average height of asperities and \( L_t \) is the total travel distance along the socket wall), the roughness considered corresponds to values of \( RF = 0.010, 0.025, 0.050, 0.085, \) and 0.106, respectively (see Fig. 5c).

Finally, to conduct the numerical analysis of the rock-socketed pile, the displacements of the boundary walls are restraint \((u_x = u_y = u_z = 0)\), see Fig. 5b) and an axial velocity \( (V_z) \) that is low enough to ensure that the model specimens remain in quasi-static equilibrium (e.g., 0.05 m/s) is applied on the wall head pile. The socket head settlement \((\delta)\) –i.e., the settlement recorded at the socket head, see Fig. 5a– and axial force \((P)\) are obtained, respectively, through the axial displacements and force reaction recorded at such wall head pile (see, Fig. 5b).

**3.2. DEM results and interpretation**

**3.2.1. Load and global stiffness-settlement response**

The load-settlement and the global stiffness –i.e., \( P/\delta \) – response from the 3D DEM simulations of piles (with different roughness) socketed into the rocks considered...
herein –sandstone (S2) and gneiss– are shown in Fig. 7. All analyses are conducted until a normalized socket head settlement of 10% of the socket diameter ($\delta/D = 10\%$) is reached. Similar to Gutiérrez-Ch et al. (2020), results in Fig. 7 show that the socket roughness is an important factor that significantly increases the load capacity and stiffness of rock-socketed piles. For instance, for piles socketed in sandstone (S2) and for $\delta/D = 2\%$, the model with $RF = 0.106$ supported a working load 13.5 times higher than that supported by the model with $RF = 0.010$, and this ratio decreases with increasing load or settlement (e.g., to 12.6 times for $\delta/D = 8\%$, see Fig. 7a). However, an opposite behavior is noted for piles socketed in gneiss (G) where the relative pile-to-rock strength ratio ($RS = \sigma_{c,\text{pile}}/\sigma_{c,\text{rock}}$) is higher (e.g., from 12.5 to 15.4 times for $\delta/D = 2\%$ and 8%, respectively, see Fig. 7b).

[Fig. 7 approx. here]

Results in Fig. 7 qualitatively agree with the load tests reported by Horvath et al. (1983), who reported an increase of the load capacity of piles socketed in shale due to socket roughness. However, as shown in Fig. 8, where our DEM results are compared to Horvath et al. (1983) load tests, these authors obtained a proportionally smaller increment than observed in numerical results (for a same $\delta/D$ and a similar increase in roughness factor). Also, results in Fig. 7 suggest that there could be an upper roughness limit beyond which the load capacity and global stiffness of the piles stops increasing. These results are in line with those of Dai et al. (2017), who reported a similar trend in small-scale pile load tests conducted at 1-g, considering socket roughness.
3.2.2. Damage at the rock-pile interface

One of the advantages of DEM simulations is being able to monitor the process of bond breakage to generate tensile and shear micro-cracks. In this work, the development of micro-cracks (shear + tensile) within the pile, and within the surrounding rock, has been recorded and plotted in Fig. 7. As it can be observed, the number of such micro-cracks is very small before a settlement of approximately $\delta/D = 1\%$, after which the number of micro-cracks increases rapidly (especially for rougher piles), leading to a more extensive damage at the rock-pile interface.

To support this result further, Fig. 9 shows the evolution of shear and tensile cracks with pile settlement for the numerical tests in sandstone (S2) and gneiss with $RF = 0.106$. As it can been seen, rock damage is very limited for $\delta/D = 1\%$ and it concentrates at the top of the model, although some cracks appear in the second asperity of the (weaker) sandstone model. (The concentration in the upper part could be due to the infinite stiffness of the wall head pile). For larger settlements ($\delta/D = 3\%$), the cracks extend downwards and penetrate into both materials, although of course mainly affecting the weaker materials in each model (rock in Fig.9a and concrete in Fig.9b). For even larger settlements ($\delta/D = 10\%$), the cracks start to become almost ubiquitous in both models. Consequently, the widely employed criterion of considering a $\delta/D = 1\%$ under design working loads is appropriate to avoid excessive damage of rock-concrete interfaces of rock-socketed piles (see e.g., Whitaker and Cooke 1966).
3.2.3. Particle motions and rotations at the rock-pile interface

Particle motions \((x, y, z)\) and rotations \((\omega_x, \omega_y, \omega_z)\) at the rock-pile interface are monitored to further evaluate the adequacy of the 2D calibration of the micro-mechanical FJCM and SJCM parameters, so that they can be used for the 3D DEM simulations of rock-socketed piles conducted herein. To do that, three tracking balls are located within the rock body, at different depths and in the vertical plane corresponding to the radial axis \(i_r\) (see Fig. 10a). Piles socketed in sandstone (S3) with \(RF = 0.025\) and 0.050 are used as study examples. (Note that, to facilitate the visualization of the tracking balls, pile particles are not shown in Fig. 10a).

Figures 10-11 show the motions and rotations of such tracking balls, until reaching a normalized socket head settlement \((\delta/D)\) of 10% (values at zero correspond to a \(\delta/D = 0\%\)). As expected in a pile, vertical motions \((z)\) are significant; however, the horizontal motions \((x, y)\) mainly occur in the radial direction \(i_r\) (see Fig. 10b) and rotations \((\omega_x, \omega_y, \omega_z)\) are not substantial (see Fig. 11), hence further supporting that the load transfer mechanism is mainly associated to dilation of the interface, just as in the 2D numerical model of the DS tests used for calibration.

Based on the results shown above, and on their discussion, the calibration of micro parameters for FJCM and SJCM using 2D models is considered appropriate, so
that such 2D calibrations can be used for 3D models. This is because: (i) there is a relatively minor influence on the macro-properties obtained, (ii) particles at the interface of a rough rock socket have radial horizontal motions, with very small particle rotations, and (iii) because they hugely reduce the computational time for the calibration process.

4. A factor $\alpha_{RF,1\%D}$ for side shear resistance prediction considering socket roughness

Next, the results of DEM models are used to estimate the mobilized average side shear resistance expected for a given socket roughness and for a given socket head settlement ($\tau_{ave,\delta}$). To that end, 15 DEM 3D simulations with different rock bodies (i.e., 5 socketed-piles in each one of the rocks considered herein – sandstones S2 and S3, and gneiss–) and with different socket roughness ($RF = 0.010, 0.025, 0.050, 0.085$ and $0.106$) have been conducted in PFC, in which the average side shear resistance of the rough rock-socketed pile has been recorded for $\delta = 1\%D$ (i.e., $\tau_{ave,1\%D}$). (Since DEM models are composed by an assembly of particles, $\tau_{ave,\delta}$ can be computed as the sum of all vertical components of contact forces acting on all particles at the rock-pile interface divided by its nominal side area.) With this information, a factor that follows the typical form of Eq. (1) has been proposed, at the same time that it goes beyond previous works, since it is able to consider the influence of the socket roughness.

In particular, a new factor, $\alpha_{RF,1\%D} = \tau_{ave,1\%D}[\text{MPa}] / \sqrt{\sigma_c[\text{MPa}]}$ is defined, and a design curve is proposed (see Fig. 12), that makes possible to estimate the
average side shear resistance in rock-socketed piles for a socket head settlement equivalent to 1% of the socket diameter. The approach considers (i) the roughness at the rock-concrete interface, through the roughness factor $RF$; and (ii) the uniaxial compressive strength, $\sigma_c$, of the weaker material (intact rock or pile). The reader should note that the proposed approach is equivalent to considering $\beta = 0.5$ in the general empirical criterion given by Eq. (1); this is for simplicity, and because it is one typical value employed by many methods that are commonly employed in practice (for a compendium of $\alpha$ and $\beta$ empirical factors see Gutiérrez-Ch et al. 2020). Therefore, the main difference with the method proposed herein is that, contrary to most previous approaches, the proposed method accounts for the socket roughness in its estimation of $\alpha$. Considering the results shown in Fig. 12, a value of $\alpha_{RF,1\%D} = 10RF$ is proposed as a lower bound for average side shear resistance prediction of piles socketed in medium rock ($\sigma_c = 12$–150 MPa).

5. Conclusions

In this paper, 2D and 3D DEM models have been developed to analyze the behavior of axially loaded rock-socketed piles considering socket roughness. Our research findings show that socket roughness is a crucial factor affecting the load capacity, side shear resistance and stiffness of rock-socketed piles, so that this parameter should be incorporated in current design practice.

This paper demonstrates that a simulation scheme based on the use of 3D DEM numerical models, with their micro-mechanical properties calibrated against
relatively simple laboratory tests (UCS and DS tests with a smooth interface), can be employed to successfully simulate rock-socketed pile behaviour. In particular, results show that they can be used (i) to reproduce their load and global stiffness settlement response, (ii) to estimate their side shear resistance for a given settlement, and (iii) to monitor the process of bond breakage and micro-crack development at the rock-pile interface.

DEM simulations conducted herein further show that a 2D calibration process is adequate (and significantly more computational efficient than a 3D calibration), so that 2D DEM models can be employed for calibration of the micro-mechanical parameters involved in the FJCM and SJCM employed herein, even when such micromechanical parameters are going to be employed in a 3D DEM simulation. Also, results demonstrated that there is a huge improvement of computational efficiency when 2D models are used during calibration of the FJCM and SJCM model parameters, even in comparison with calibrations conducted using 3D models with periodic boundary conditions.

Finally, the results of this research have been employed to propose a factor, 
\[ \alpha_{RF,1\%D} = \tau_{ave,1\%D}/\sqrt{\sigma_c}, \]
 to estimate the average side shear resistance of rock sockets under an approximate socket head settlement of 1% of the socket diameter. Although further validation of the methodology with additional good-quality load tests data is necessary, a preliminary design curve is also proposed, with results suggesting that a value of \( \alpha_{RF,1\%D} = 10RF \) – where \( RF \) is the roughness factor originally defined by Horvath et al. (1983) – might be an adequate lower bound to be employed to estimate the average side shear resistance of pile.
socketed in medium rock, with UCS values in the range of, approximately, 12–150 MPa.

6. Acknowledgement

This research was funded, in part, by the Spanish Ministry of Economy, Industry and Competitiveness under current BIA 2015-69152-R. The first author has received during 2019 the scholarship for PhD research provided by the José Entrecanales Ibarra Foundation. Both support is gratefully acknowledged.

7. References


List of Tables

**Table 1.** Average values of rock properties.

**Table 2.** Particle and flat-joint micro-mechanical properties fitted from the UCS tests employed in the calibration of intact materials in 2D DEM.

**Table 3.** Micro-mechanical properties of SJCM calibrated on planar interface with 2D DEM direct shear tests.

**Table 4.** Macro-properties fitted for the UCS tests employed for calibration of intact Sandstone-S3 in 2D and 3D DEM.

**Table 5.** Computational time for modelling UCS test of intact sandstone (S3) in 2D and 3D DEM.

**Table 6.** Computational time for DS tests conducted on unbonded concrete-sandstone (S3) planar joint until a limit shear displacement of 1 mm.
Table 1. Average values of rock properties

<table>
<thead>
<tr>
<th>Sample</th>
<th>Macro-properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>1Gneiss (G)</td>
<td>$\sigma_c$ (MPa)</td>
</tr>
<tr>
<td></td>
<td>$E$ (GPa)</td>
</tr>
<tr>
<td></td>
<td>$\nu$</td>
</tr>
<tr>
<td>1Concrete (C2)</td>
<td>$\sigma_c$ (MPa)</td>
</tr>
<tr>
<td></td>
<td>$E$ (GPa)</td>
</tr>
<tr>
<td></td>
<td>$\nu$</td>
</tr>
<tr>
<td>2Sandstone (S2)</td>
<td>$\sigma_c$ (MPa)</td>
</tr>
<tr>
<td></td>
<td>$E$ (GPa)</td>
</tr>
<tr>
<td></td>
<td>$\nu$</td>
</tr>
<tr>
<td>2Sandstone (S3)</td>
<td>$\sigma_c$ (MPa)</td>
</tr>
<tr>
<td></td>
<td>$E$ (GPa)</td>
</tr>
<tr>
<td></td>
<td>$\nu$</td>
</tr>
<tr>
<td>2Concrete (C1)</td>
<td>$\sigma_c$ (MPa)</td>
</tr>
<tr>
<td></td>
<td>$E$ (GPa)</td>
</tr>
<tr>
<td></td>
<td>$\nu$</td>
</tr>
</tbody>
</table>

Source: Data from 1Gutiérrez (2013), 2Gu et al. (2003)
Table 2. Particle and flat-joint micro-mechanical properties fitted from the UCS tests employed in the calibration of intact materials in 2D DEM

<table>
<thead>
<tr>
<th>Micro-mechanical properties</th>
<th>Sandstone-S2</th>
<th>Sandstone-S3</th>
<th>Concrete-C1</th>
<th>Gneiss</th>
<th>Concrete-C2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective modulus of both particle and bond, $E^* = E^\circ$ (GPa)</td>
<td>1.70</td>
<td>1.90</td>
<td>27.00</td>
<td>43.00</td>
<td>26.20</td>
</tr>
<tr>
<td>Normal-to-shear stiffness ratio of both particle and bond, $k_n^* / k_s^* = k_n^\circ / k_s^\circ$</td>
<td>1.35</td>
<td>1.45</td>
<td>2.75</td>
<td>3.25</td>
<td>3.00</td>
</tr>
<tr>
<td>Friction angle, $\theta$ (°)</td>
<td>32</td>
<td>35</td>
<td>30</td>
<td>35</td>
<td>30</td>
</tr>
<tr>
<td>Ball density, $\rho$ (kg/m$^3$)</td>
<td>2500</td>
<td>2550</td>
<td>2500</td>
<td>2690</td>
<td>2500</td>
</tr>
<tr>
<td>Minimum radius, $R_{\text{min}}$ (mm)$^a$</td>
<td>1.0</td>
<td>1.0</td>
<td>0.8</td>
<td>0.8</td>
<td>1.0</td>
</tr>
<tr>
<td>Maximum radius, $R_{\text{max}}$ (mm)$^a$</td>
<td>1.4</td>
<td>1.4</td>
<td>1.2</td>
<td>1.2</td>
<td>1.4</td>
</tr>
<tr>
<td>Cohesion, $c$ (MPa)</td>
<td>4.85</td>
<td>7.90</td>
<td>13.55</td>
<td>40.80</td>
<td>18.80</td>
</tr>
<tr>
<td>Tensile strength, $\sigma_t$ (MPa)</td>
<td>2.8</td>
<td>3.5</td>
<td>6.0</td>
<td>20.0</td>
<td>9.0</td>
</tr>
</tbody>
</table>

Source: Data from Gutiérrez-Ch et al. (2018, 2020). $^a$Based on a sensitivity analysis not reproduce herein.
**Table 3.** Micro-mechanical properties of SJCM calibrated on planar interface with 2D DEM direct shear tests

<table>
<thead>
<tr>
<th>SJCM parameters</th>
<th>Sandstone(S2)-concrete(C1)</th>
<th>Sandstone(S3)-concrete(C1)</th>
<th>Gneiss-concrete(C2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint normal stiffness, $k_{n,SJ}$ (MPa/mm)</td>
<td>10</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Joint shear stiffness, $k_{s,SJ}$ (MPa/mm)</td>
<td>5</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Joint coefficient of friction, $\mu_{SJ} (\tan \emptyset)$</td>
<td>0.70</td>
<td>0.70</td>
<td>0.60</td>
</tr>
</tbody>
</table>

Source: Data from Gutiérrez-Ch et al. (2018, 2020)
Table 4. Macro-properties fitted for the UCS tests employed for calibration of intact Sandstone-S3 in 2D and 3D DEM

<table>
<thead>
<tr>
<th>Macro-properties</th>
<th>Experimental data by Gu et al. (2003)</th>
<th>2D DEM</th>
<th>3D DEM</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average COV (%)</td>
<td>Average COV (%)</td>
<td>Average COV (%)</td>
</tr>
<tr>
<td>( \sigma_c ) (MPa) ± SD</td>
<td>(21.77 ± 1.19) ((n = 8))</td>
<td>(21.65 ± 1.43) ((n = 10))</td>
<td>(24.66 ± 0.28) ((n = 10))</td>
</tr>
<tr>
<td>( E ) (GPa) ± SD</td>
<td>(3.25 ± 0.39) ((n = 5))</td>
<td>(2.32 ± 0.02) ((n = 10))</td>
<td>(2.35 ± 0.01) ((n = 10))</td>
</tr>
<tr>
<td>( \nu ) ± SD</td>
<td>(0.1 ± 0.00) ((n = 3))</td>
<td>(0.11 ± 0.01) ((n = 10))</td>
<td>(0.10 ± 0.0) ((n = 10))</td>
</tr>
</tbody>
</table>

SD = Standard Deviation, COV = coefficients of variation, \(n\) = number of tests
Table 5. Computational time for modelling UCS test of intact sandstone (S3) in 2D and 3D DEM

<table>
<thead>
<tr>
<th>System Information</th>
<th>Time of 2D DEM (1,145 particles)</th>
<th>Time 3D DEM (23,036 particles)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intel Core i7-8700K CPU @ 3.7 GHz, RAM 32 Gb</td>
<td>1.5 minutes</td>
<td>125 minutes</td>
</tr>
</tbody>
</table>
Table 6. Computational time for DS tests conducted on unbonded concrete-sandstone (S3) planar joint until a limit shear displacement of 1 mm

<table>
<thead>
<tr>
<th>System information</th>
<th>Time 2D DEM (12,241 particles)</th>
<th>Time 3D DEM, PBC (32,559 particles)</th>
<th>Time 3D DEM (81,337 particles)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intel Core i7-8700K CPU @ 3.7 GHz 3.7 GHz, RAM 32 Gb</td>
<td>2.1 hours</td>
<td>18.5 hours</td>
<td>50 hours</td>
</tr>
<tr>
<td>Joint length / Joint width</td>
<td>400 mm / [N.A.], 200 mm / 10 mm, 200 mm / 25 mm</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Fig. 1. Calibration of FJCM using UCS tests with 2D DEM (1,145 particles) and 3D DEM (23,036 particles): (a) sandstone (S3) specimens, (b) total displacement vectors at the end of test, (c) axial stress-strain curves.
Fig. 2. DEM specimens for DS tests on unbonded concrete-sandstone (S3) planar interfaces CNL conditions: (a) 2D DEM models (12,241 particles), (b) 3D DEM models (81,337 particles) DEM models, PBC (32,559 particles).
Fig. 3. Comparison of DS tests results conducted on unbonded concrete-sandstone (S3) planar joint with different normal stresses $\sigma_n$ (laboratory data from Gu et al. 2003).
Fig. 4. Motions of tracking balls for a DS test conducted on unbonded concrete-sandstone (S3) planar joint under $\sigma_n = 400$ kPa with PBC: (a) locations of tracking balls (red balls), (b) x,y,z motions vs. shear displacement curves.
Fig. 5. (a) rock-socketed pile and nomenclature, (b) 3D DEM rock-socketed pile model, (c) 2D view of the socket roughness at the rock-pile interfaces for models in 3D DEM, and asperity geometry for model pile with $RF = 0.106$. 

Particle size distribution
1. $R_{min} = 1 \text{ cm}$
2. $R_{min} = 1.5 \text{ cm}$
3. $R_{min} = 2.25 \text{ cm}$

Top view of the walls boundary condition
$u_x = u_y = u_z = 0$
Fig. 6. Selection of the minimum radius ($R_{\text{min}}$) of the particles of the models with 3D DEM: sensitivity analysis performed on piles socketed into sandstone (S3) with $RF = 0.025$ (to facilitate the illustration only the number of particles corresponding to the body of the pile is represented).
Fig. 7. Load-settlement and global stiffness-settlement curves for piles socketed in sandstone (S2) and gneiss (G) considering different degree of socket roughness.
Fig. 8. Comparison of load-settlement response of piles socketed in sandstone (S2) computed with DEM and piles socketed in shale reported in load tests by Horvath et al. (1983).
Fig. 9. Micro-crack evolution with settlement for piles socketed in sandstone (S2) and gneiss with $RF = 0.106$. 
Fig. 10. Motions at the rock-pile interface for pile socketed in sandstone (S3) with $RF = 0.025$ and $0.050$. 
Fig. 11. Rotations at the rock-pile interface for pile socketed in sandstone (S3) with RF = 0.025 and 0.050.
Fig. 12. Factor $\alpha_{RF, 1\%D}$ for average side shear resistance $\tau_{ave, 1\%D}$ prediction considering a socket head settlement equivalent to $1\%D$. 

$$\frac{\tau_{ave,1\%D}}{\sqrt{\sigma_c}} = 10RF$$