Discontinuum Analysis for Tunnelling Purposes: Procedure and Considerations for Creating 2D Models by Applying the FDEM Method

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ABSTRACT
By applying the Finite-Discrete Element Method (FDEM) in this study, modelling assumptions, including considerations and limitations, such as the application of 2D methods to simulate the 3D effects of tunnel advancement, the effect of the out-of-plane stress etc., will be addressed. Additionally specific suggestions will be provided in order to assist with the setup of FDEM tunnel models in hard rockmasses and serve as a guide to the practicing engineer for tunnelling projects. The specifics associated with such a modelling methodology/strategy, model set-up, calibration and validation (and considerations at each stage) are the focus, with the paper also summarizing in a systematic fashion, the subsequent checks required of a tunnel design engineer in order to gain confidence with the numerical model as well as the analysis process for an underground tunnel excavation.

RÉSUMÉ
En appliquant la méthode d'éléments discrets finis (FDEM), incluant des hypothèses de modélisation, des considérations et des limitations, telles que l'applications des méthodes 2D pour une simulation d'effets en 3D de l'avancement d'un tunnel, l'effet du stress hors-plan, etc., sera adressé pour la mise en place des modèles de tunnels FDEM dans les masses rocheuses. De plus, ceci servira d'un guide pour les projets de tunneling à l'ingénieur en exercice. Les spécificités d'une telle méthodologie/stratégie de modélisation, la configuration, l'étalonnage et les validations d'un modèle (et les considérations durant chaque étape) sont les mises au point de ce document. Ce document met également l'accent de manière systématique les contrôles ultérieurs exigés d'un ingénieur de conception de tunnel afin que celui-ci obtienne plus de confiance avec le modèle numérique et le processus d'analyse pour une excavation de tunnel souterrain.

1 INTRODUCTION

Underground excavations are becoming more and more common as alternative infrastructure 'spaces' as civil surface infrastructure becomes more congested and constrained. These spaces, serving multiple purposes, including: transportation, mining, containment of hazardous contaminants, storage of valuable resources etc. have become more complex due to modern society's requirements and as a result of the increasing demand in underground development. As such, numerous projects take place at greater depths under high magnitude stresses and within more competent, hard rockmasses.

Numerical modelling has proven a valuable tool for the design process of such projects as it allows the design engineers to simulate various conditions and take into consideration multiple scenarios in order to obtain a better insight of the rockmass response during the excavation process and to assist in optimizing the employed design. Developed underground excavation and tunnel numerical models in most cases are based on traditional failure criteria such as the Mohr-Coulomb or the Hoek-Brown (Hoek et al. 2002) which depend on the peak shear strength of a material to predict its response during an excavation process. Work by various researchers (Vlachopoulos 2009, Vlachopoulos et al. 2013, Oke et al. 2014a, Langford et al. 2015) have shown the applicability of such an approach within weak rockmasses (i.e. heavily fractured, disturbed, and soft rocks) in which high magnitude displacements are expected. Under these rockmass conditions shearing is the dominant failure mechanism and the numerical analysis results are in agreement with relevant field observations.

However, within hard, competent, massive, highly interlocked rockmasses at low confinement environments around a tunnel excavation boundary, the material response in not driven by shear failure. On the contrary, brittle failure associated with hard rock underground excavations is well documented (Kaiser and McCreath 1994, Lee et al. 2004, Diederichs et al. 2004) and the material response is governed by the tensile strength of the rockmass, with the medium failing in extension due to high magnitude compressive induced stresses as a result of the excavation (Diederichs 2003, Diederichs 2007). Therefore, conventional shear based failure criteria are not appropriate for capturing this rockmass brittle behaviour (Vlachopoulos and Vazaios 2018) and other numerical techniques are more appropriate for such instances. Different approaches for the numerical simulation of brittle failure include the use of continuum modelling and the application of a cohesion weakening friction strengthening (CWFS) model (Hajiabdolmajid et al. 2002, Diederichs 2007, Perras and Diederichs 2016), discontinuum modelling based on the discrete element method (DEM) (Potyondy and Cundall 2004, Shin 2010, Farahmand et al. 2017), and the hybrid finite-discrete element method (FDEM) (Mahabadi 2012, Lisjak et al. 2015, Vazaios et al. 2018)

Following the work by Vazaios et al. (2018), and Vlachopoulos and Vazaios (2018), the modelling procedure of a two-dimensional (2D) numerical model for the simulation of a tunnel excavation at great depths in a massive rockmass by applying the FDEM method is discussed herein. The tunnel scale model is built in the numerical package Irazu (Geomechanica Inc. 2017) and various aspects of the model are examined in order to provide guidelines for the setup of a numerical model at a tunnel scale.

2 THE FDEM METHOD

The combined finite-discrete element method (FDEM) merges finite element tools and techniques with discrete element algorithms in order to capture the fracturing processes. More specifically, finite-element based analysis of continua is combined with discrete element-based transient dynamics, contact detection and contact interaction solutions. The numerical model is comprised of large number of deformable bodies that may interact with one another and in this process they can break, fracture or fragment (Munjiza 2004). Due to the capability of the method to allow for the dynamic simulation of multiple interacting objects, a simulation can begin with either a single intact domain or a collection of discrete intact bodies. For the creation of a tunnel scale model the first approach is applied as discussed in the next sections.

Within the FDEM method, which accommodates the finite strain elasticity coupled with a smeared crack model for the simulation of discontinuous systems, the deformation of the bulk material is captured by 3-noded, linear-elastic, constant-strain, triangular elements with the impenetrability enforced by a penalty function and the continuity constrained by bonding forces of interface elements in between the triangular elements (Munjiza et al. 1999), as depicted in Figure 1.

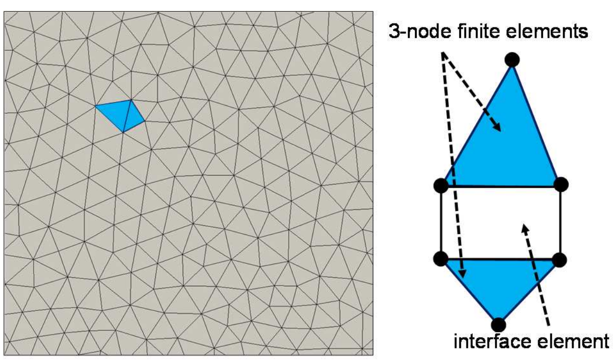

Figure 1. Representation of 2D medium using an unstructured mesh of 3-noded, triangular elements linked by 4-node interface elements.

In order to simulate the fracturing and the progressive failure of rock materials, a cohesive-zone approach is adopted. By employing this technique, the strength degradation of the interface elements allows for the progressive failure of rocks (Lisjak et al. 2015), and therefore, a macroscopic constitutive model (e.g. Hoek-Brown) is not required. The main advantage of this approach is that the fracturing process only depends on induced stresses and strains and the crack trajectories do not need to be determined a priori. They are, however, controlled by the mesh topology. The peak shear strength of the interface elements can be expressed as a function of the internal friction angle φ, cohesion c, and the normal stress σ_n. Once the peak shear strength is exceeded, the interface element transitions to a softening mode until the assigned fracture energy in shear G_{II} is depleted, followed by the removal of the "broken" element. In similar fashion, the interface elements are assigned a strength in tension, and when the peak tensile strength f_t is exceeded, the interface element "softens" until its fracture energy in tension G_I is consumed. The reader is referred to Mahabadi (2012), Lisjak (2013), and Tatone and Grasselli (2015) for more information.

3 MODEL SETUP

Brittle fracturing within hard rock excavations is controlled by extensile cracks along the direction of high magnitude compressive stresses forming around the tunnel boundary as the in situ stress field changes due to the underground opening. Vazaios et al. (2018) demonstrated the potential of the FDEM method to capture these complex failure mechanisms under low confining stresses by using the well documented case of the Underground Research Laboratory (URL) Test Tunnel (Martin et al. 1997), located in Pinawa, Manitoba in Canada. The numerical model that was generated for this purpose is illustrated in Figure 2, and hereafter it will be used in this paper as the reference model in order to discuss the features, considerations and limitations of this modelling approach for deep, underground, hard rock excavations.

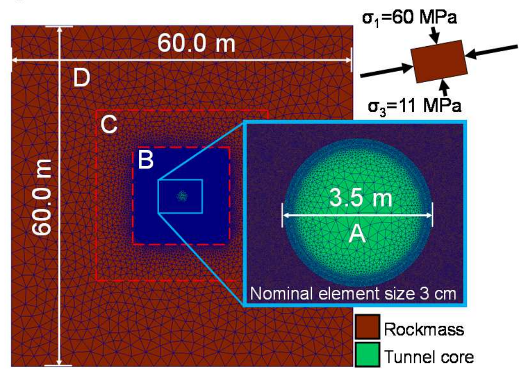

Figure 2. Tunnel model configuration for the URL Test Tunnel created in Irazu. The model is divided into four different areas A, B, C and D with the element size being 0.03-0.5 m, 0.03 m (constant size employed), 0.03-1.5m, and 1.5-2.5 m respectively (Vazaios et al. 2018).

3.1 Element size

In continuum numerical techniques, usually smaller element sizes result in higher accuracy results. However, as noted by Diederichs (2007), for the simulation of brittle processes, small element sizes may lead to unrealistic stress and strain localization, with a coarser mesh of higher order elements being more preferable to capture the high stress gradients of the lower order elements of a finer mesh.

Unlike continuum approaches, in order to capture the brittle fracturing processes of hard rock excavations, the adopted element size should be small enough relative to the scale of the model in order to secure that the generated fracture pattern is going to be relatively independent of the adopted mesh configuration and that the fracturing mechanisms are adequately captured (Gao and Stead 2014, Farahmand and Diederichs 2015, Tatone and Grasselli 2015). For the reference model adopted by Vazaios et al. (2018), the selected element size was approximately 2% of the tunnel radius R=1.75 m (i.e. 0.03 m) which is considered adequate for the simulation of brittle fracturing. This element size was uniform and was adopted for only a smaller sub-domain of the numerical model within which material fracturing was expected. In this domain of interest, it is strongly advised that a graded mesh be avoided and a mesh with a uniform element size is required. Larger element sizes as one is moving towards the model boundaries (similar technique as that used within continuum models) can be adopted and out of the area of inters a graded mesh can be used (Figure 2).

3.2 External boundary and boundary conditions

In most geomechanics problems, a semi-infinite medium (i.e. the ground surrounding an excavation) is involved, however, the FDEM method, similar to the finite element method (FEM) and the discrete element method (DEM) requires a finite computational domain. Therefore, the need for artificial far-field boundary conditions arises (Mahabadi 2012).

For tunnel models using the FEM method, pins and rollers are typically used in order to simulate the far field conditions depending on the specific requirements of the project. For example, for the simulation of a tunnel close to the ground surface where the vertical displacement needs to be captured, a free boundary is assigned to the ground surface, rollers (i.e. zero horizontal displacement) are assigned to the sides and pins (i.e. zero horizontal and vertical displacements) at the bottom of the tunnel. On the contrary, for a deep tunnel there is no such requirement and usually pins are utilized along the external boundary.

In a similar fashion within the FDEM method, the external boundaries are assigned a displacement condition for the simulation of the far field conditions. However, in dynamic problems, these boundary conditions may cause unrealistic reflections of outward propagation stress waves (Mahabadi 2012). One way to prevent boundary effects is by extending the domain boundaries far enough away. This is similar to techniques used in FEM tunnel models in which the plastic zone surrounding the opening dictates the external boundaries (Oke et al. 2014b). However, due to the high speed of elastic waves in rock materials, this solution is often computationally impractical. A valid alternative integrated into Irazu (Geomechanica Inc. 2017) is the use of an absorbing (non-reflective) boundary condition which allows for the necessary energy dissipation. Based on the solution proposed by Lysmer and Kuhlemeyer (1969), viscous boundary tractions are used to numerically absorb the kinetic energy of incident waves (Mahabadi 2012). Based on the work by Lisjak (2013), Vazaios et al. (2018), and Vlachopoulos and Vazaios (2018), the ratio between the width of the external boundary and the diameter of the opening (*W/D*) should be at least between 10 and 15, and an absorbing boundary condition should be applied.

3.3 Field Stresses

The final stress state, deformation, stability conditions and the potential failure mode within the vicinity of an underground opening depends on the in situ stress distribution and magnitude. In the FDEM method, the specified stress state is translated into nodal forces that gradually deform the finite element mesh until a static equilibrium has been achieved. These assigned stresses form the initial conditions of the rockmass prior to the excavation.

For tunnelling projects, and especially for deep tunnels in situ stresses are frequently initialized by assigning a uniform Cauchy stress tensor in the entirety of the modelling domain (deep tunnel assumption). In this way, gravity induced stress gradients are neglected, which are more appropriate for a tunnels closer to the ground surface.

3.4 Analysis Method for Capturing 3D Effects in 2D

Since the effect of an excavation in a rockmass is 3D in nature, within 2D plain strain analyses, the progressive displacement of the tunnel boundary must be recreated in order to replicate the gradual loss of confinement due to the excavation sequence.

Within the Irazu software, the face replacement method is used in order to replicate the 3D effect. Plane strain simulation of tunnel advance in this method involves the replacement of the tunnel core with unstressed, elastic material of reduced modulus during each step. In this way the tunnel boundary is allowed to converge during the subsequent model step until the stresses reestablish in the tunnel core and a temporary equilibrium is reached (Vlachopoulos and Diederichs 2014).

The reduction of the modulus is performed in a linear fashion over a number of steps until the complete removal of the excavation material. Since the excavation within the FDEM method is a dynamic process, the user must ensure that a large number of steps is employed in order to secure that pseudo-static stress conditions are maintained at each stage of the excavation and dynamic oscillations are avoided. If the number of steps in not adequate for the simulation of the excavation, then dynamic effects may occur which may result in unrealistic fracturing processes and damage extents.

3.5 Selection of Penalty Values

Without the presence of the interface elements, the emergent tangent elastic modulus and Poisson's ratio measured in an unconfined compression strength (UCS) test should match the input values (Tatone and Grasselli 2015). However, due to the presence of the interface elements that are necessary for the simulation of the fracturing processes, the overall effective stiffness of the system is reduced as a result of the finite stiffness of these elements and the infinitesimal inter-element penetrations. To maintain the correct emergent elastic behaviour (Figure 3) appropriate values of the Young's modulus, Poisson's ratio, and the penalty terms used have to be selected. While large penalty term values ensure that the linear elastic response is achieved, smaller time steps are required in order to maintain numerical stability during the analysis. Therefore, a balance between computational cost and the desired material response is required.

In order to achieve the emergent elastic properties of a given rock material, Tatone and Grasselli (2015) used the modulus and Poisson's ratio obtained from laboratory testing as input and then calibrated the three penalty terms (normal contact penalty, tangential contact penalty, and fracture penalty). On the other hand, in terms of the simulation of the URL Test Tunnel, Vazaios et al. (2018) made an assumption that the penalty terms are an order of magnitude mode that Young's modulus and calibrated the modulus and Poisson's ratio based on that assumption. This ensured that the already small time step (due to the element size) would not have to be further decreased.

3.6 Selection of Strength Parameters

Having determined the penalty terms and the deformability parameters of the numerical model, the parameters controlling the fracturing process of the material can then be calibrated. Tatone and Grasselli (2015) suggest that cohesion *c*, friction angle *φ*, and tensile strength f_t can be used initially directly from laboratory testing results and the fracture energy parameters are determined in order to recreate the observed failure mechanisms in UCS and Brazilian numerical tests. Furthermore, it is noted that multiple combinations of fracture parameters can yield the same peak macroscopic strength.

For a tunnel scale model, however, this calibration can be more complicated as the in situ rockmass strength is almost always lower than the strength obtained in the lab. For the simulation of the URL Test Tunnel, Vazaios et al. (2018) initially attempted to calibrate a large scale UCS and Brazilian test in order to determine a set of parameters yielding the same UCS and indirect tensile strength as the massive granite of the URL (Martin 1994). However, once these strength parameters were applied in a tunnel scale model, no fracturing was observed (Vazaios and Vlachopoulos 2017), hence, indicating that reduced strength parameter values should be used.

The second phase of the calibration process focused on using the laboratory test results for *c*, *φ*, and f_t and adjust G_I and G_{II} to capture the v-shaped notch observed at the URL Test Tunnel. Slight adjustments of the *c*, *φ*, and f_t allowed for the replication of both the damage extent and the failure mechanism (spalling) within the numerical model based on field observations, as shown in Figure 3. By using these established strength parameters within a UCS, numerical test showed that the UCS strength of the rockmass in situ is almost half of what reported from lab testing (Figure 4); a finding consistent with other researchers (Hajiabdolmajid et al. 2002, Potyondy and Cundall 2004, Cai and Kaiser 2014). In a similar fashion, Lisjak et al. (2015) also reported that calibration of a tunnel scale model based on laboratory tests results does not yield results consistent with in situ observations and calibration of the tunnel scale model has to be performed based on material response observed in the field.

3.7 Damping Coefficient

For quasi-static problems, such as a tunnel excavation, a critical damping factor should be used in order to dissipate unwanted dynamic oscillations. Higher values of the damping coefficient suppress high-frequency stress waves, hence, resulting in the in the replication of quasi-

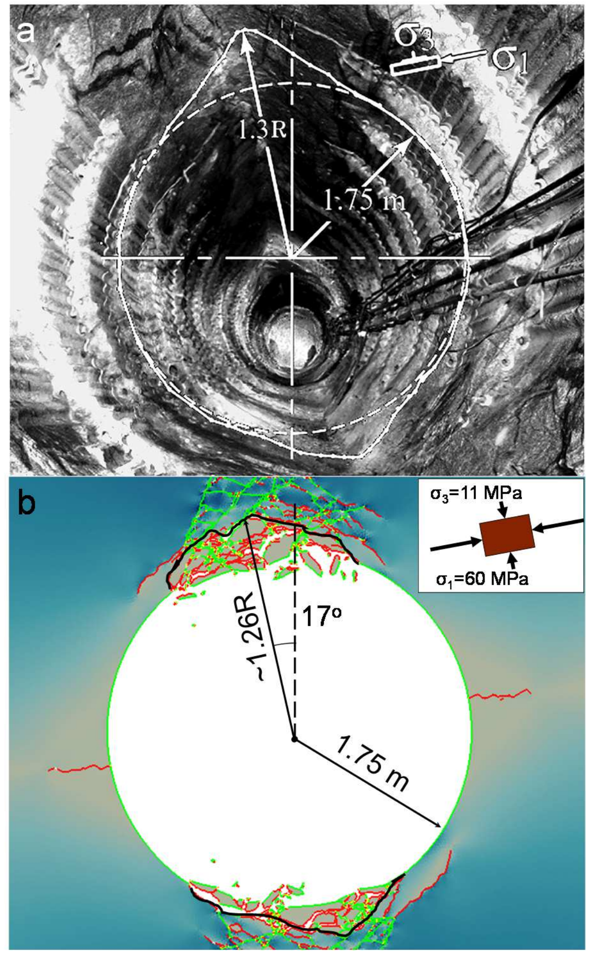

Figure 3. (a) Photograph of the URL Test Tunnel (after Diederichs 2007) showing the damage profile observed in situ. (b) Damage profile from the FDEM model (highlighted black) after the completion of the numerical analysis. Fractures in extension (Mode I) are red and fractures in shear (Mode II) are green (Vazaios et al. 2018).

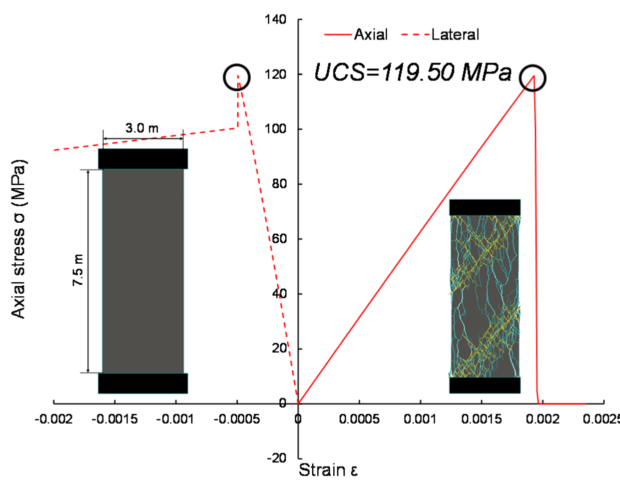

Figure 4. Stress-strain curves obtained from the UCS test using calibrated strength parameters for the interface elements (Vazaios et al. 2018).

-static conditions (Tatone and Grasselli 2015). However, by increasing significantly the viscous damping, the time step size decreases and therefore a balance needs to be maintained as it affects the total computation time.

3.8 Timestep and Analysis Steps

Having established the aforementioned parameters of the analysis for a tunnel scale model, the final step is to determine the required timestep size and the number of required time steps to run the analysis.

If the timestep size is too big, the model will not run or will undergo numerical instability. On the contrary, very small timesteps result in very long computational times if results are to be obtained. The timestep size depends on several factors which include: the element size (larger elements result in larger timesteps while smaller elements in smaller timesteps), the density of the material (by increasing the material density the timestep size increases), the Young modulus of the material (large Young moduli values result in smaller timesteps), the viscous damping (increase in damping requires smaller timesteps), and the penalty terms (increased values of the penalty terms decrease the required timestep). In order to achieve an optimal timestep, a trial-and-error process is required depending on the model requirements and the simulated materials.

Once the timestep size has been established, the number of time steps for the analysis have to be determined. Typical simulations require from a few hundreds of thousands to several million time steps. The number of time steps required depends on the timestep size, as an analysis with a larger timestep size requires less steps than an analysis with a smaller timestep size until accurate results are obtained. Based on the work by Vazaios et al. (2018), and Vlachopoulos and Vazaios (2018) for a timestep size between 4×10^{-8} s and 6×10^{-8} s a number between 300,000 to 600,000 for the initialization of the geostatic stress state, 1,500,000 to 3,000,000 steps for the excavation process, and 500,000 to 700,000 for the establishment of equilibrium after the complete removal of the tunnel core material appears to be adequate for a tunnel scale simulation in brittle rocks. Additionally, the expected damage extent around the excavation should be taken into account as well in order to determine the number of time steps required.

4 MODEL VERIFICATION

After the model has been generated, the user has to ensure that that the input and run parameters are appropriate and that they simulate the required field conditions. In this section, a verification process of the model is discussed in order to provide selected preliminary guidelines for the user.

4.1 Geostatic Stress State

Similar to tunnel scale models using the FEM method, prior to the excavation, the in situ stresses have to be initialized. In nature, an undisturbed rockmass is under static equilibrium conditions which means that the field stresses are balanced and the rockmass remains undeformed unless the in situ stress regime changes. Therefore, in the numerical model the initial stresses have to reflect the in situ stress state, and after their initialization the resulting displacements and velocities have to be of a very low magnitude approximating zero. In an FDEM model, an adequate number of steps during the establishment of the geostatic stresses is important in order to create the required field stress regime (Figure 5).

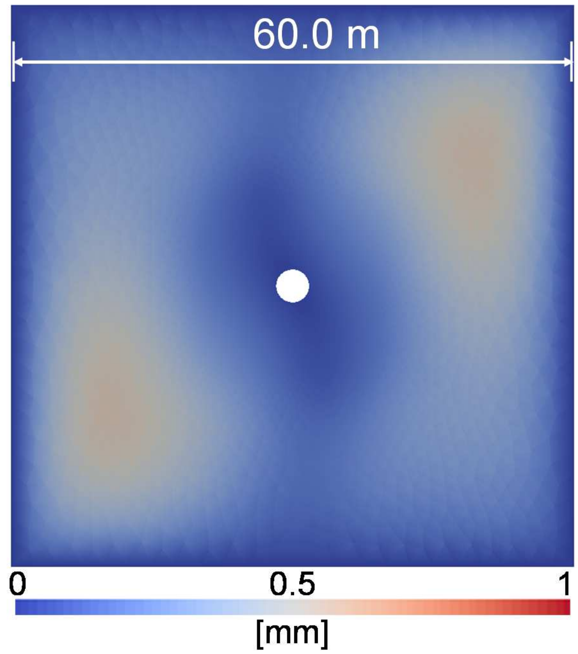

Figure 5. Displacement magnitude contours at the completion of the geostatic stage of the model.

4.2 Model Elastic Response

During the analysis and by assuming that no-fracturing occurs, the FDEM model should be able to reproduce the results of a FEM analysis given the same conditions, as shown in Figure 6. This ensures that the model when behaving elastically produces the same results as well established solutions.

4.3 Fracturing of Interface Elements

A properly calibrated model will fracture and form a damaged zone surrounding the underground opening, as shown in Figure 2. The fracturing process depends on the assigned strength parameters of the interface elements (as discussed in the previous sections) which once exceeded, results in the formation and propagation of cracks. In brittle rockmasses the dominant failure mode of the interface elements is in tension due to the extensile state that the material undergoes as a result of the high compressive major principal stresses as the tunnel advances. In Figure 7, it can be observed the deconfinement that the rockmass undergoes at the crown of the excavation and the resulting failure due to tension.

5 MODEL VALIDATION

Following the calibration and verification of the FDEM tunnel scale model, the obtained results have to be compared to the results of established solutions in order to ensure the validity of the model and its results. Numerical models developed using the FEM method are usually compared to the results of established solutions in order to

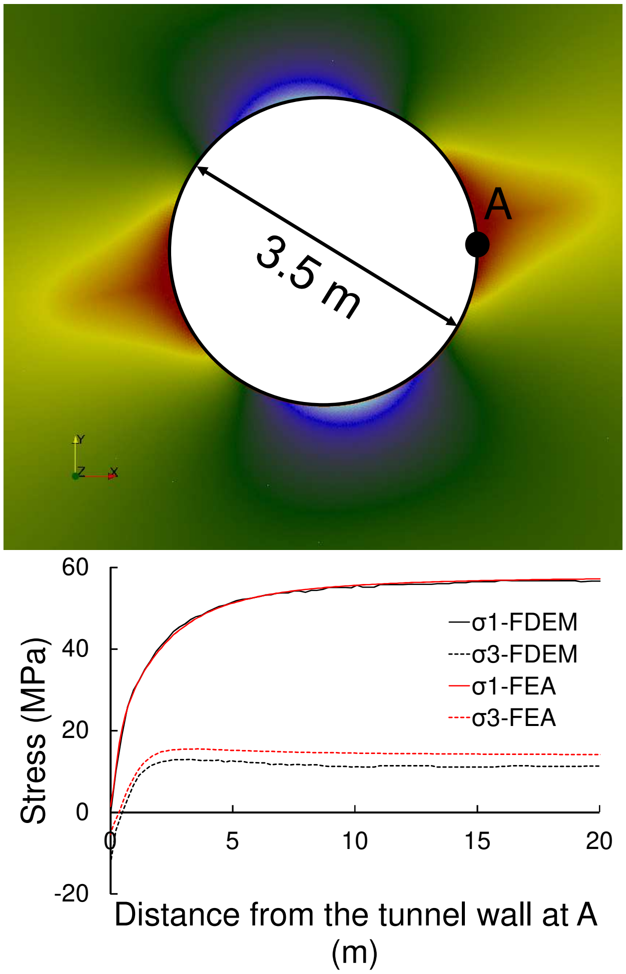

Figure 6. Comparison between FDEM and FEM principal stress results. The measurements were taken along a horizontal line starting at point A (Vazaios and Vlachopoulos 2018).

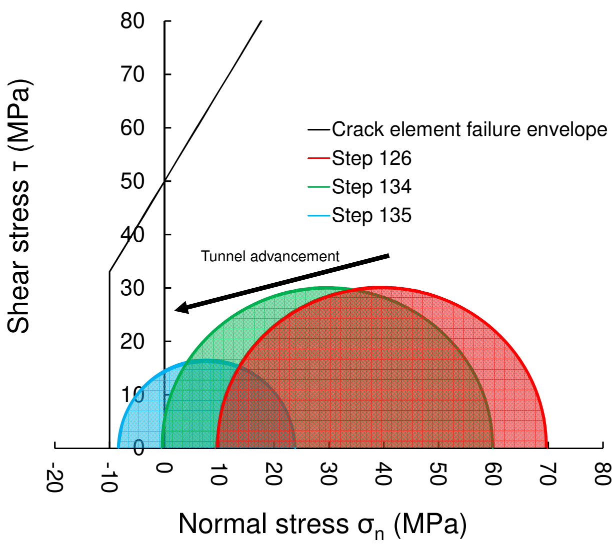

Figure 7. Crack elements failure envelope and Mohr circles of a single crack element failing in extension.

ensure the validity of the model and its results. Numerical models developed using the FEM method are usually compared to well-established analytical solutions and field observations in order to validate the results. Especially for weak rockmasses, shear based failure criteria integrated into FEM models have produced good results (Vlachopoulos et al. 2013, Oke et al. 2014a, Langford et al. 2015) which are validated from analytical solutions as well. However, that is not the case for brittle failure as the extensile fracturing involves complex failure mechanisms that are hard to capture based on analytical methods.

While analytical solutions for the brittle failure of hard rocks in tunnel excavations are not readily available, well-established continuum modelling methods like the CWFS model (Hajiabdolmajid et al. 2002) or the damage initiation spalling limit (DISL) method (Diederichs 2007) can be used for the simulation of brittle fracturing in massive rockmasses, and their results can be compared to the FDEM model results. Vlachopoulos and Vazaios (2018) demonstrated that the results between the continuum approach of the DISL method and the discontinuum FDEM model for the simulation of the URL Test Tunnel are in good agreement; mostly regarding the extent of the HDZ (Figure 8). Additionally, acoustic emission (AE) and micro-seismic (MS) events recorded at the URL Test Tunnel are in good agreement with the FDEM results. It has to be noted that the calibration of the FDEM model using the URL field observations was based on the simulation of the HDZ. The EDZ obtained in the numerical model is an emergent feature showing that its calibration results in a good representation of the material in situ. Additionally, the results of large scale UCS models (Figure 4) yield similar results to other researchers' work (Vazaios et al. 2018).

For the modelling of hard rock excavations by applying the FDEM method, the validation of the model can be performed by using well-established methodologies of brittle failure based on continuum models, with an emphasis on massive, hard rockmasses. Validation of the numerical model can also be performed based on field observations given a specific site, if suitable field data is available.

6 CONSIDERATIONS AND LIMITATIONS

6.1 Mesh Sensitivity

The calibration of an FDEM model is performed for a given element size that has to be small enough in order to be able to capture the fracturing processes in brittle rocks as discussed earlier. Furthermore, numerical methods utilizing a mesh approach for the discretization of the modelling domain have an inherent sensitivity to the mesh topology, which may affect the fracture trajectories. In order to overcome this, a random discretization scheme should be adopted. Mahabadi et al. (2012) and Lisjak et al. (2014) suggest the use of an unstructured Delaunay triangulation scheme that should be applied in order to minimize the constraints imposed by the mesh configuration.

By maintaining the same input parameters but varying the element size, an investigation was conducted in order to examine its effect on the tunnel scale model response. From Figure 9, it can be seen that by transitioning from an element size of 0.03 m to 0.05 m and finally to 0.10 m, the system response is becoming stiffer and stronger due to the penetration of discrete bodies and the critical openings increase with increasing of the element size (Mahabadi 2012). As observed, the stronger and stiffer configurations suppress the extensile cracks and therefore, not only do they not replicate the damage extent observed in the field but they also fail to replicate the extensile failure mechanism in brittle rocks. Therefore, it becomes evident that the calibrated strength parameters for one element size cannot be used for another, and the calibration process has to be repeated for the given element size.

6.2 Density Scaling

When DEM modelling approaches are employed for the simulation of geomaterials, the required computational time required poses a major problem for the numerical simulation, especially for system with a large number of discrete bodies. As previously discussed, in the FDEM method, a density increase results in increased timestep sizes, and therefore, this may result in less time steps required to obtain results and subsequently less computational time. Researchers by using other DEM methods (Thornton 2000, O'Sullivan and Bray 2004) used a density scaling approach in order to reduce the computational time. For quasi-static problems this could be an appropriate way to shorten simulation times (Zhao 2011). However, such a technique should be used with caution as it may affect significantly the simulation process and yield unrealistic results if very high density values are used.

6.3 Out-of-Plane Stress

Within the 2D FDEM method, the crack elements do not account for the influence of the out-of-plane stress component of the in situ stresses. Therefore, during the simulation, fracture nucleation and growth are determined only by the in-plane stresses, and the deconfinement effect of the tunnel core softening. In this way, however, the impact of the intermediate principal stress on the rockmass strength is neglected, and its effect should be taken into account through the proper calibration of the input strength parameters based on field observations.

7 SUMMARY

The numerical simulation of hard rock excavations can be particularly complicated and challenging, as a result of the complexity of the failure mechanisms that are required to be captured. In this study, a comprehensive overview of the different components which have to be taken into consideration for the numerical modelling of deep underground excavations within hard, massive rockmasses using the FDEM method was presented and discussed. Furthermore, a set of specific guidelines for the initial model setup and determination of its input parameters was provided (Figure 10). The authors would like to note that the development of a numerical model is tied to the in situ specific conditions of a project and the field observations should guide and update the numerical model as required.

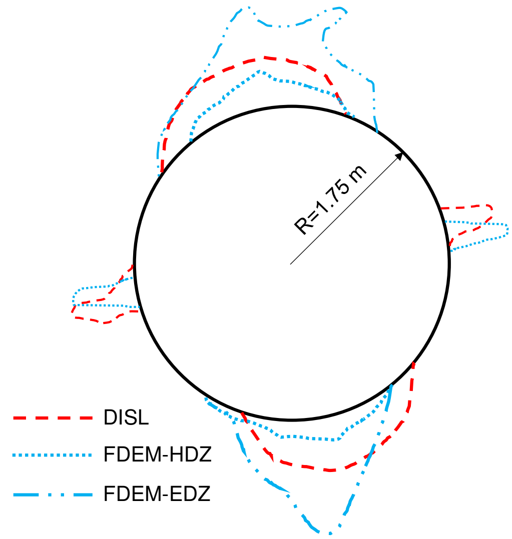

Figure 8. Damage profiles obtained from the FDEM model and the FEM model using the DISL approach (modified after Vlachopoulos and Vazaios 2018).

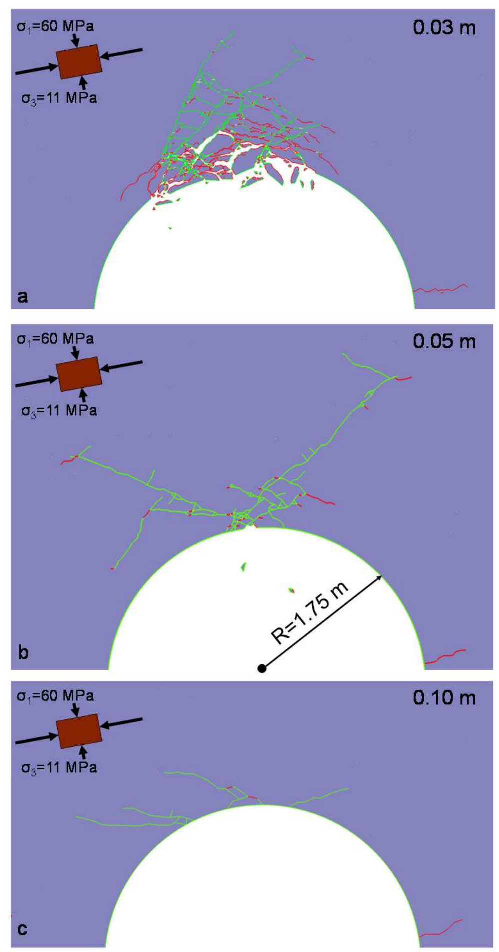

Figure 9. Failure mechanisms and damage extent for the general model configuration shown in Figure 2 but different nominal element sizes for the refined area of the model: (a) 0.03 m, (b) 0.05 m, and (c) 0.10 m. Tensile fractures are red and shear fractures are green.

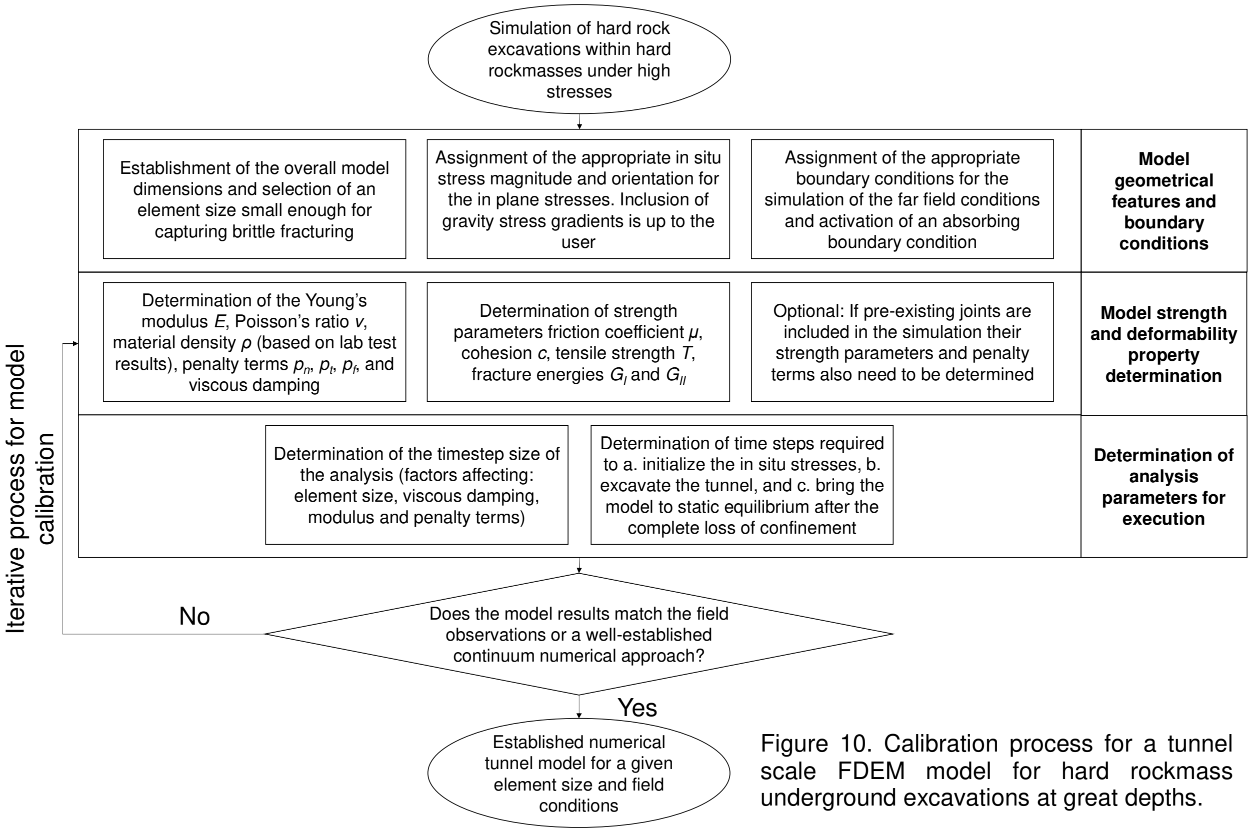

Figure 10. Calibration process for a tunnel scale FDEM model for hard rockmass underground excavations at great depths.

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REFERENCES

Cai, M., and Kaiser, P.K. 2014. In-situ rock spalling strength near excavation boundaries. *Rock Mechanics and Rock Engineering*, 47(2), pp. 659-675.

Diederichs, M.S. 2003. Manuel Rocha medal recipient rock fracture and collapse under low confinement conditions. *Rock Mechanics and Rock Engineering*, 36(5), pp.339-381.

Diederichs, M.S., Kaiser, P.K., and Eberhardt, E. 2004. Damage initiation and propagation in hard rock during tunnelling and the influence of near-face stress rotation. *International Journal of Rock Mechanics and Mining Sciences*, 41(5), pp. 785-812.

Diederichs, M.S. 2007. The 2003 Canadian Geotechnical Colloquium: Mechanistic interpretation and practical application of damage and spalling prediction criteria for deep tunnelling. *Canadian Geotechnical Journal*, 44(9), pp.1082-1116.

Farahmand, K., and Diederichs, M.S. 2015. A calibrated Synthetic Rock Mass (SRM) model for simulating crack growth in granitic rock considering grain scale heterogeneity of polycrystalline rock. In *49th US Rock Mechanics/Geomechanics Symposium*. American Rock Mechanics Association.

Farahmand, K., Vazaios, I., Diederichs, M.S., and Vlachopoulos, N. 2017. Investigating the scale-dependency of the geometrical and mechanical properties of a moderately jointed rock using a synthetic rock mass (SRM) approach. *Computers and Geotechnics*, 95: 162-179.

Gao, F.Q., and Stead, D.O.U.G. 2014. The application of a modified Voronoi logic to brittle fracture modelling at the laboratory and field scale. *International Journal of Rock Mechanics and Mining Sciences*, 68, pp. 1-14.

Geomechanica Inc., 2017. Irazu 2D Geomechanical Simulation Software, version 3.0

Hajiabdolmajid, V., Kaiser, P.K., and Martin, C.D. 2002. Modelling brittle failure of rock. *International Journal of Rock Mechanics and Mining Sciences*, 39(6), pp. 731-741.

Hoek, E., Carranza-Torres, C. and Corkum, B. 2002. Hoek-Brown failure criterion-2002 edition. *5th North American Rock Mechanics Symposium and 17th Tunneling Association of Canada Conference*: NARMS-TAC, 2002, pp. 267-271.

Kaiser, P.K., and McCreath, D.R. 1994. Rock mechanics considerations for drilled or bored excavations in hard rock. *Tunnelling and Underground Space Technology*, 9(4), pp. 425-437.

Langford, J.C., Vlachopoulos, N., and Diederichs, M.S. 2015. Revisiting Support Optimization at the Driskos Tunnel using a Quantitative Risk Approach. *Journal of Rock Mechanics and Geotechnical Engineering*, 8(2): 147-163.

Lee, S.M., Park, B.S., and Lee, S.W. 2004. Analysis of rockbursts that have occurred in a waterway tunnel in Korea. *International Journal of Rock Mechanics and Mining Sciences*, 41, pp. 911-916.

Lisjak, A. 2013. *Investigating the influence of mechanical anisotropy on the fracturing behaviour of brittle clay shales with application to deep geological repositories*. Ph.D. thesis, University of Toronto, Toronto, Canada.

Lisjak, A., Grasselli, G., and Vietor, T. 2014. Continuum–discontinuum analysis of failure mechanisms around unsupported circular excavations in anisotropic clay shales. *International Journal of Rock Mechanics and Mining Sciences*. 65: 96–115.

Lisjak, A., Garitte, B., Grasselli, G., Müller, H.R., and Vietor, T. 2015. The excavation of a circular tunnel in a bedded argillaceous rock (Opalinus Clay): short-term rock mass response and FDEM numerical analysis. *Tunnelling and Underground Space Technology*, 45, pp. 227-248.

Lysmer, J., and Kuhlemeyer, R. 1969. Finite dynamic model for infinite media. *Journal of the Engineering Mechanics Division, Proceedings ASCE*, 95(EM4): 859-876.

Mahabadi, O.K. 2012. *Investigating the influence of micro-scale heterogeneity and microstructure on the failure and mechanical behaviour of geomaterials*. Ph.D. thesis. University of Toronto. Toronto, Canada.

Mahabadi, O.K., Lisjak, A., Grasselli, G., and Munjiza, A. 2012. Y-Geo: a new combined finite-discrete element numerical code for geomechanical applications. *International Journal of Geomechanics*, 12: 676–688.

Martin, C.D. 1994. *The strength of massive Lac du Bonnet granite around underground openings*. Ph.D. thesis, University of Manitoba, Winnipeg, Manitoba, Canada.

Martin, C.D., Read, R.S., and Martino, J.B. 1997. Observations of brittle failure around a circular test tunnel. *International Journal of Rock Mechanics and Mining Sciences*, 34(7): 1065-1073.

Munjiza, A. 2004. *The Combined Finite-Discrete Element Method*. John Wiley & Sons Ltd., Chichester, West Sussex, England.

Munjiza, A., Andrews, K.R.F., and White, J.K. 1999. Combined single and smeared crack model in combined finite-discrete element analysis. *International Journal for Numerical Methods in Engineering*: 44, 41–57.

Oke, J., N .Vlachopoulos, and M.S Diederichs 2014a. Numerical Analyses in the Design of Umbrella Arch Systems. *Journal of Rock Mechanics and Geotechnical Engineering*, 6(6): 546-564.

Oke, J., Vlachopoulos, N., and Vazaios, I. 2014b. Procedure and consideration when creating a geomechanical three dimensional numerical analysis for construction of tunnel infrastructure. In *the Proceedings of the GeoRegina 2014 Conference*, Regina, Canada.

O'Sullivan, C., and Bray, J.D., 2004. Selecting a suitable time step for discrete element simulations that use the central difference time integration scheme. *Engineering Computations*, 21(2/3/4), pp.278-303.

Perras, M.A., and Diederichs, M.S. 2016. Predicting excavation damage zone depths in brittle rocks. *Journal of Rock Mechanics and Geotechnical Engineering*, 8: 60-74.

Potyondy, D.O., and Cundall, P.A. 2004. A bonded-particle model for rock. *International Journal of Rock Mechanics and Mining Sciences*, 41(8), pp. 1329-1364.

Shin, S.W. 2010. *Excavation disturbed zone in Lac du Bonnet granite*. PhD Thesis, University of Alberta, Edmonton, Alberta.

Tatone, B.S.A, and Grasselli, G. 2015. A calibration procedure for two-dimensional laboratory-scale hybrid finite-discrete element simulations. *International Journal of Rock Mechanics and Mining Sciences*, 75: 56-72.

Thornton, C., 2000. Numerical simulations of deviatoric shear deformation of granular media. *Géotechnique*, 50(1), pp.43-53.

Vazaios, I. and Vlachopoulos, N. 2017. The evolution of the excavation damage zone of tunnels in brittle rockmasses using a FEM/DEM Approach. In the *Proceedings of the 70th Canadian Geotechnical Conference, GeoOttawa 2017*, Ottawa, Canada

Vazaios, I., Vlachopoulos, N., and Diederichs, M.S. 2018. The mechanical analysis and interpretation of the EDZ formation around deep tunnels within massive rockmasses using a hybrid finite-discrete element approach: The case of the AECL URL Test Tunnel. *Canadian Geotechnical Journal* (Accepted April 4th 2018).

Vlachopoulos, N. 2009.*Back analysis of a tunnelling case study in weak rock of the Alpine System in Northern Greece: Validation and optimization of design analysis based on ground characterization and numerical simulation*. Ph.D. thesis, Queen's University, Kingston, Ontario, Canada.

Vlachopoulos, N., Diederichs M.S., Marinos, V. and Marinos P. 2013. Tunnel Behaviour associated with the weak Alpine rock masses of the Driskos Twin Tunnel system, Egnatia Odos Highway. *Canadian Geotechnical Journal*, 2013, 50(1): 91-120.

Vlachopoulos, N., and Diederichs, M.S. 2014. Appropriate uses and practical limitations of 2D numerical analysis of tunnels and tunnel support response. *Geotechnical and Geological Engineering*, 32(2), pp. 469-488.

Vlachopoulos, N. and Vazaios, I. 2018. The numerical simulation of hard rocks for tunnelling purposes at great depths: A comparison between the hybrid FDEM method and continuous techniques. Advances in Civil Engineering, pgs. 18.

Zhao, X.L., 2011. Numerical modeling of granular assembly using discrete element method. *Advanced Materials Research*, 146, pp. 738-742.