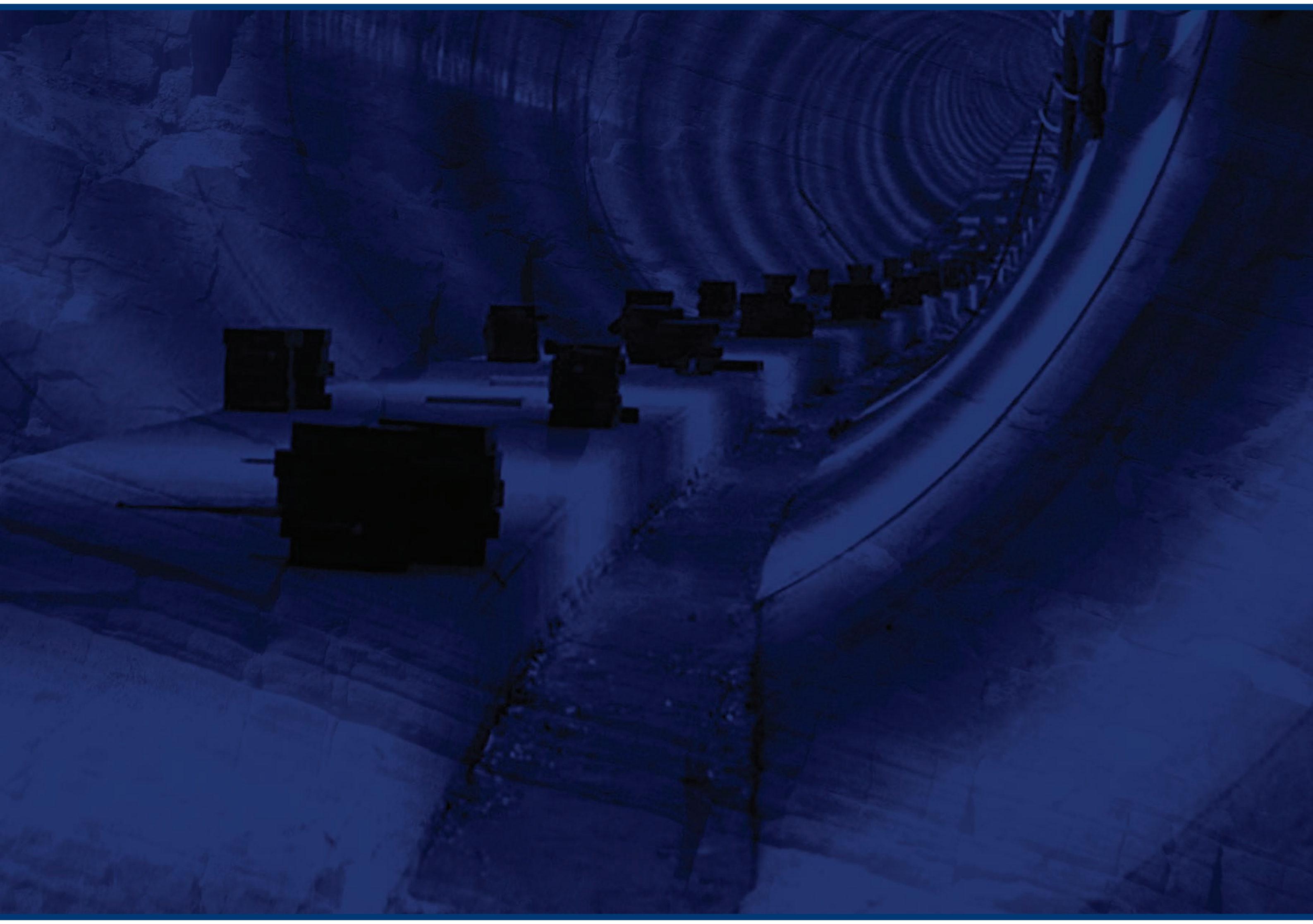
PROCEEDINGS

2018 AUSTRALIAN GEOMECHANICS SOCIETY VICTORIAN SYMPOSIUM

Geotechnics and transport infrastructure

Wednesday, 24 October 2018, 8:00am – 6:00pm
Rydges Hotel, 186 Exhibition Street, Melbourne



AUSTRALIAN GEOMECHANICS SOCIETY
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PREFACE

The Victorian chapter of the Australian Geomechanics Society invited academics and practitioners in the field of geotechnical and ground engineering to attend the 2018 Australian Geomechanics Society Victorian Symposium on 'Geotechnics and transport infrastructure' held on 24 October 2018.

In recent years Victoria has seen significant investment in transport infrastructure as part of a plan to manage the demands of a growing population and expanding urban fringe. The construction of Melbourne Metro, a second crossing of the Yarra River, rail and freeway upgrades as well as numerous level crossing removal projects are just some of the major transport projects currently underway in Melbourne and regional Victoria. Many of these projects carry numerous complex geotechnical challenges.

The 2018 Australian Geomechanics Society Victorian Symposium covers a variety of geotechnical challenges associated with transport geotechnics and present overviews of current infrastructure challenges, state of the-art practices, innovation, new research results and case studies demonstrating applications of advanced techniques and cost effective solutions in the construction and design of local transport infrastructure. The Symposium brought together professional engineers, researchers, specialist contractors, regulators, educators and students to share and discuss their experiences on the topic of transport infrastructure and associated geotechnical challenges and applications.

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AUSTRALIAN GEOMECHANICS SOCIETY

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Keynote address

Recent advances in the design of Australia’s transport infrastructure: an overview of the ARC Centre of Excellence for Geotechnical Science and Engineering Activities

G. Kouretzis, S. W. Sloan and O. Buzzi

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ABSTRACT

The ARC Centre of Excellence for Geotechnical Science and Engineering (CGSE) was established in 2011 to pioneer new scientific approaches for the design of Australia’s energy and transport infrastructure. This paper presents an overview of selected CGSE projects relevant to testing and modelling of soft soils, the development of computational methods for the failure analysis of geotechnical infrastructure, and the assessment and mitigation of rockfall hazard. The first part is dedicated to findings from Australia’s first National Field Testing Facility for soft soils, established by the CGSE in Ballina, NSW. Emphasis is put on i) the geotechnical characterisation of soft estuarine clay deposits, ii) the performance of a full-scale trial embankment on soil improved with prefabricated vertical drains, and iii) outcomes of an international symposium on predicting the behaviour of embankments on soft soils. The second part discusses the use of the Finite Element Limit Analysis method to estimate efficiently and accurately the collapse load of geostructures, and the application of this method to determine the required internal tunnel support pressure to prevent collapse. The paper concludes with the presentation of novel numerical and experimental methods to model rockfalls and to assess the performance of protective systems in the laboratory and *in situ*.

Keywords: embankments, soft soils, finite element limit analysis, rockfalls

1 INTRODUCTION

The Australian Research Council (ARC) Centre of Excellence for Geotechnical Science and Engineering (CGSE) is a collaborative research centre that capitalised on the combined strengths of the Priority Research Centre for Geotechnical Science and Engineering at the University of Newcastle, the Centre for Offshore Foundation Systems at the University of Western Australia and the Centre for Geotechnics and Railway Engineering at the University of Wollongong. The total CGSE funding was approximately $23M over a seven year period (2011-2018) and, apart from the ARC, the research activities of the CGSE were supported by its industry partners (Douglas Partners, Coffey Geotechnics and Fugro) and also by Roads and Maritime Services (RMS) NSW, the Office of the Chief Scientist and Engineer NSW and the Ballina Bypass Alliance.

Apart from providing a national focus for geotechnical research by integrating the complimentary expertise of three key Australian geotechnical research groups into a single unit, the goals of the CGSE were to: i) Optimise the design of critical infrastructure by combining fundamental work in geotechnical science, cutting edge computational modelling, state-of-the-art physical modelling and field testing to make the engineering of Australia’s infrastructure safer and more cost-effective; ii) Educate and train the next generation of geotechnical engineers and researchers; iii) Collaborate with the offshore and onshore industry to ensure a rapid uptake of geotechnologies and design practices (cgse.edu.au)

This paper presents a brief overview of selected CGSE projects relevant to transport infrastructure, and is structured as follows: The first part, dedicated to testing and modelling of soft soils, summarises findings of laboratory, *in situ* and full-scale tests performed at the National Soft Soil Field Testing Facility (NFTF), established by the CGSE near Ballina, NSW. The second part presents in brief the Finite Element Limit Analysis method, developed for the Ultimate Limit State analysis of geostructures, and its application to determine the required internal tunnel support pressure to prevent collapse. The final part presents numerical and experimental methods developed in the CGSE to model rock fragmentation during rockfalls and to assess the performance of relevant protective systems.

2 BALLINA SOFT SOIL FIELD TESTING FACILITY

2.1 General

The establishment of a soft soil testing facility by the CGSE near Ballina, NSW was motivated by the difficulties encountered during the construction of a nearby section of the Pacific Highway: Embankments with maximum fill height 14m settled up to 6.4m over a period of 3 years, while accurate prediction of settlement magnitude and rate of evolution of embankment deformations proved to be challenging (Kelly 2014). The reason is that these fills were built on estuarine soft clay deposits, commonly found along the eastern and southern Australian coastlines. Australian high- to extremely high plasticity estuarine clays exhibit low undrained shear strength and high compressibility, and are characterised by presence of electrolytes in the pore fluid, traces of organic matter and expansive minerals as well as weak cementation. All these features render modelling of their mechanical behaviour particularly demanding.

The establishment of the NFTF has allowed thorough *in situ* investigations to be combined with advanced laboratory tests on high-quality samples to characterise a representative Australian estuarine soft clay. In addition, two full-scale trial embankments were constructed in 2013

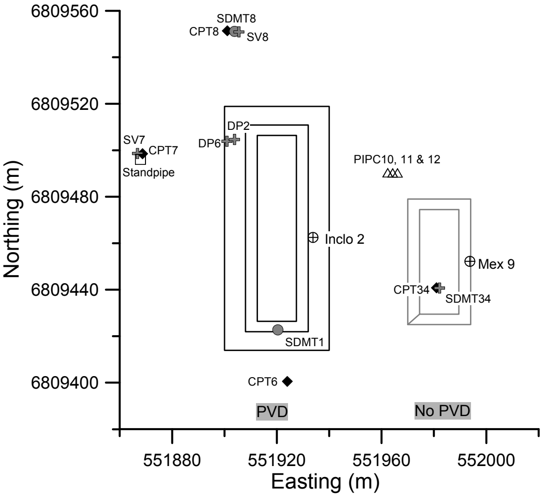


Figure 1. Plan view of the NFTF at Ballina, NSW.

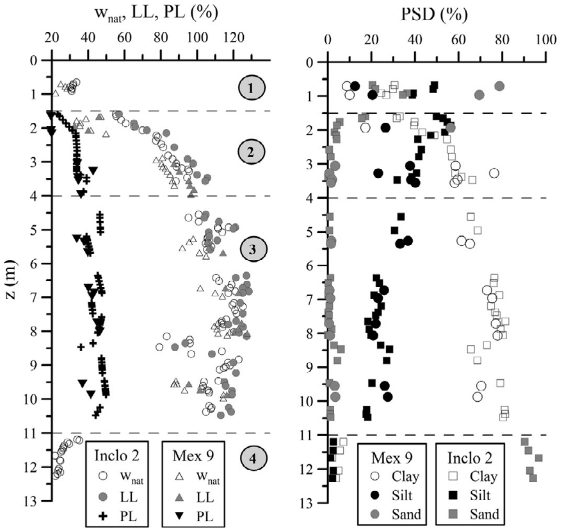


Figure 2. Variation of index properties with depth.

(Kelly et al. 2017a, 2018), one on subsoil improved with both conventional wick drains and biodegradable jute drains (PVD, Fig. 1). The following subsections summarise findings from the laboratory and *in situ* characterisation studies that are of key importance for compiling a detailed geotechnical model of the soft estuarine silty clay deposits, referred to hereinafter as Ballina clay. Next, elements of the performance of the trial embankment are presented in short, together with a discussion on certain key aspects of soil behaviour that need to be properly captured in an analysis model to simulate the behaviour of embankments on soft soils.

2.2 Soft soil characterisation study

The subsoil at the NFTF comprises an approximately 0.2m thick surficial layer of organic material (decomposing sugar cane plants) underlain by a 1.0m to 1.3m thick layer of sandy to clayey silt alluvium deposited during flood events (Layer 1, Fig. 2). Soft estuarine clay deposits (Holocene age) under the alluvium mantle infill the Richmond river valley in northern NSW, and are about 9m thick at the site. The upper estuarine silty clay layer (Layer 2, Fig. 2) is characterised by increasing plasticity, void ratio and electrical conductivity with depth (the last two quantities are not included in Fig. 2) while the lower estuarine silty clay layer (Layer 3, Fig. 2) has nearly constant soil state properties. A transition zone (Layer 4, Fig. 2) approximately 4m thick lies below the estuarine deposits, turning into a fine sand layer that extends up to a depth of about 18m-19m. The groundwater table level fluctuates between the base of the alluvial silt layer (-1.5m) and 0.5m above ground level, due to standing water after heavy rainfall.

Index properties, as well as mechanical parameters, were obtained from laboratory tests performed on tube specimens retrieved from two continuous boreholes (Inclo 2 and Mex 9, Fig. 2). Samples were retrieved from depths up to 13 m using an Osterberg-type fixed piston sampler (89 mm external diameter). The characterisation study included: (i) the inspection of tube specimens with non-destructive imaging techniques, (ii) index characterisation testing, and (iii) advanced characterisation testing. The basic characterisation tests focused on determining index properties, composition, and fundamental parameters of the natural soil deposits. Advanced characterisation tests were carried out to determine the variation of the hydraulic and mechanical properties along the soil profile. A detailed description of the laboratory characterisation methodology can be found in Pineda et al. (2016). In addition, a comprehensive *in situ* testing campaign (Kelly et al. 2017b) took place in parallel with the laboratory element tests, consisting of geophysics, cone penetrometer, seismic dilatometer, vane shear, piezocone (CPTu) and BAT permeability tests. Selected results from both laboratory and *in situ* tests are presented and discussed in the following, while the complete data set has been made openly available to the geotechnical community through the CGSE website (cgse.edu.au) and the Datamap application (Doherty et al. 2018).

Figure 2 summarises the main index properties obtained from the laboratory tests. Ballina clay has an organic content of around 3% and soil activity equal to 1. Its main mineral components are kaolinite, illite, quartz, illite/smectite and amorphous minerals. The natural water content increases with depth from 20% up to 120%. Differences between the liquid limit and natural water content are less than 10-15%. The plastic limit ranges between 20% and 53%, whereas the liquid limit varies from 55% to 135%. The dry density ρ_d reduces from 1.50Mg/m^3 to 0.70Mg/m^3 with depth, and the minimum variation in ρ_d occurs between 3m and 11m depth. Particle size distributions (Fig. 2) exhibit some discrepancies between the two boreholes Inclo 2 and Mex 9 at shallow depths (z<2m), mainly in terms of sand content. The clay content is predominant below 2m, with maximum values of up to 82%, while the sand content lies around 1%. The marine nature of Ballina clay implies that the influence of the pore fluid salinity is significant and this was considered during mechanical testing. Specifically, the electrical conductivity of the pore fluid increases with depth from 5mS/cm ($z \approx 1$m) up to 36.5mS/cm at around z=8m.

The initial vertical effective stress σ'_{v0}, as well as estimates of the effective vertical yield stress σ'_{yield} (preconsolidation pressure) obtained from Constant Rate of Strain (CRS) tests, are plotted with depth in Fig. 3. Both uncorrected and corrected (for rate effects) values are presented in this figure. Solid lines represent profiles interpreted by Kelly et al. (2018). A yield stress ratio YSR (overconsolidation ratio, OCR) of around 1.4 was obtained for Ballina clay based on the results shown in Fig. 3. Coefficients of vertical consolidation c_v (CRS tests) and horizontal consolidation c_h, (CPTu dissipation tests) are also shown in Fig. 3. c_v values corresponding to the initial stress and the yield stress are depicted with different symbols. The small differences between c_h and c_v, suggest that anisotropy of the permeability of Ballina clay is rather small.

Undrained shear strength was evaluated from laboratory and *in situ* tests. An increase in the undrained shear strength (triaxial compression and extension tests) with depth is observed in Fig. 3. The ratio between the undrained shear strength in compression (TC) and extension (TE) tests is approximately 0.66, which indicates some degree of strength anisotropy that lies in the range reported in the literature for natural soft clays. Values of undrained shear strength interpreted from *in situ* tests (CPTu, SDMT and FV) are also shown in Fig. 3. The CPT and SDMT data have been calibrated on the FV data, and the N_{kt} factor used to convert the CPT data to FV strength was 13.2.

2.3 Trial embankment on PVD-improved soil and conclusions from the Embankment Prediction Symposium

Following retrieval of the soil samples and *in situ* tests, the construction of the trial PVD embankment commenced in July 2013 and was completed within about 2 months. A typical cross-section of its geometry is presented in Fig. 4. The crest of the embankment is 80m long by 15 m wide and the nominal inclination of the batters is 1.5H:1V. A 95m long by 25m wide working platform was initially extended below the transition zone, into the fine sand layer (Layer 4, Fig. 2). An extensive instrument network constructed, followed by the placement of a sand drainage layer beneath the footprint of the embankment. Layers of separation geofabric were placed above and below the sand drainage layer. The average density of the fill from nuclear density tests was about 2Mg/m^3, therefore the applied load on the ground surface is of the order of 60kPa. Vertical drains were installed on a nominally 1.2m square grid from the top of the working platform, and was deployed to monitor the performance of the embankment during and after construction, consisting of vibrating wire piezometers (VWP), total pressure cells, magnetic extensiometers, hydrostatic profile gauges, inclinometers, push-in pressure cells and settlement plates to record settlement of the ground surface below the embankment.

The factual geotechnical data together with details regarding the construction of the embankment were made available to practitioners and Australian and international academics, who attempted to predict the performance of the embankment over a 3-year period. Accordingly, a numerical prediction symposium was held in Newcastle on 12 and 13 September 2016, which attracted 28 Class A predictions (Kelly et al. 2018).

Comparison of measurements of embankment settlement with the predictions is shown in Fig. 5. Generally, settlement was under-predicted and the rate of settlement was over-predicted. The symposium proved that, despite being a classical problem in geotechnical engineering, it is difficult to accurately predict the behaviour of embankments on soft soils, and that the current state of practice in Australia is broadly consistent with that of industry and academic contributors from abroad.

The divergence between predictions and measurements was largely attributed to the parameters selected by predictors to describe the mechanical behaviour of Ballina clay (Kelly et al. 2018). Predictors seemed to be unused to having such an amount of high-quality field and laboratory test data, and tended to rely on experience from past projects rather than facts when choosing the key mechanical parameters. For example, it appears that the influence of strain rate on the key mechanical parameters of soft clays, such as the preconsolidation pressure and undrained shear strength, is a concept not well rooted in geotechnical practice. Indeed, rate effects were not considered in most of the predictions, perhaps due to the lack of experience in geotechnical practice with the analysis of CRS tests. Figure 6 compares the range of yield stress ratio YSR values adopted in the predictions with those estimated from CRS tests, after correction of the yield stresses for rate effects. The profile interpreted by Kelly et al. (2018) is also included in this figure for comparison. It is clear from Fig. 6 that many predictors chose relatively high values for the YSR, perhaps because they were not familiar with the interpretation of CRS tests. Many predictors merely adopted uncorrected values, which contributed to the underprediction of the total settlement.

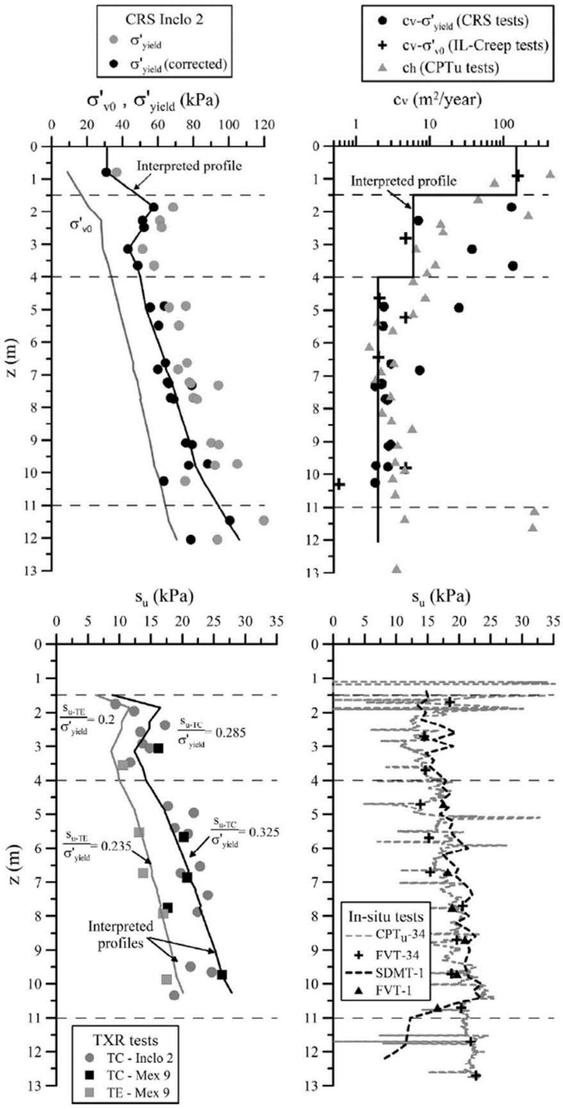


Figure 3. Variation of mechanical parameters with depth.

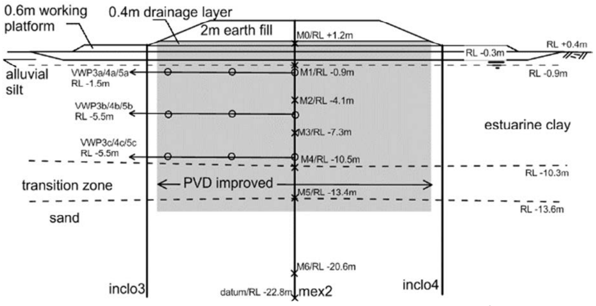


Figure 4. Embankment section and locations of indicative instruments.

The permeabilities adopted by many predictors were typically overestimated by an order of magnitude, although values for the soft clay layer ranged across almost 3 orders of magnitude as shown in Fig. 6. Some predictors adopted constant values of permeability, without considering its drop as the void ratio decreases. The reason for this apparent discrepancy is not clear, but it is perhaps due to a conventional rule of thumb that the *in situ* permeability is an order of magnitude larger than the value measured in the laboratory. Disturbance during conventional sampling and testing frequently affects the quality of laboratory test data, and thus designers often resort to their experience and questionable rules of thumb to obtain analysis parameters. The adoption of high values of permeability results in higher rates of settlement and pore pressure dissipation than is measured in the field.

3 FINITE ELEMENT LIMIT ANALYSIS

A highly effective method for the solution of stability problems in geomechanics is known as the Finite Element Limit Analysis (FELA) method, which has been pioneered at the Newcastle node of the CGSE (Sloan 2013). Based on finite elements, the limit theorems of classical plasticity theory and large scale optimisation, FELA permits the exact collapse load to be bracketed from above and below, thus providing an in-built error indicator. The lower bound formulation of FELA computes a statically admissible stress field which satisfies equilibrium, the yield criterion, and the stress boundary conditions. Its upper bound counterpart, on the other hand, determines a kinematically admissible velocity field which satisfies the plastic flow rule and velocity boundary conditions. Both the lower bound and upper bound formulations lead to large-scale nonlinear optimisation problems which are solved very efficiently using bespoke interior-point algorithms. Unlike traditional finite element methods, the grids in FELA are naturally discontinuous, with velocity/stress discontinuities being present between adjacent elements in the upper/lower bound formulations. This renders the method ideal for Ultimate Limit State analyses, where finite element codes suffer from numerical issues relevant to mesh distortion and poor convergence. The latest version of the FELA software can handle problems with inhomogeneous soil profiles, anisotropic strength characteristics, soil-structure interaction and three-dimensional geometries. Moreover, it incorporates robust mesh adaptivity schemes, 3D geometries, the effects of steady-state pore water pressures, and a strength reduction process for computing the safety factor in terms of strength.

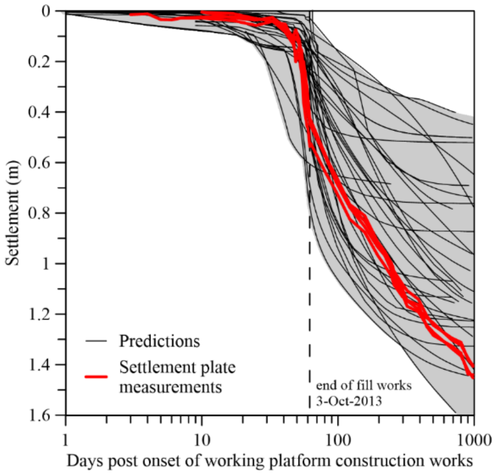


Figure 5. Ground surface settlement versus predictions of performance.

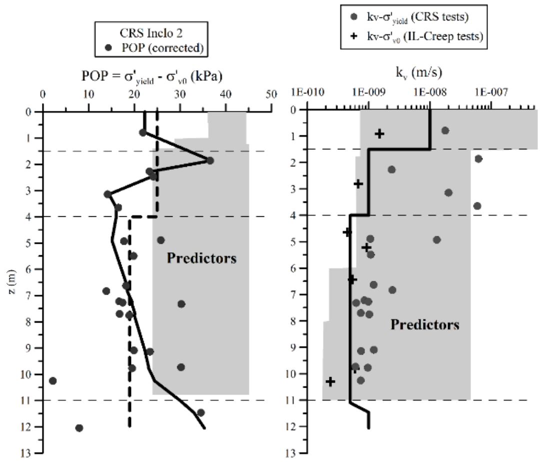
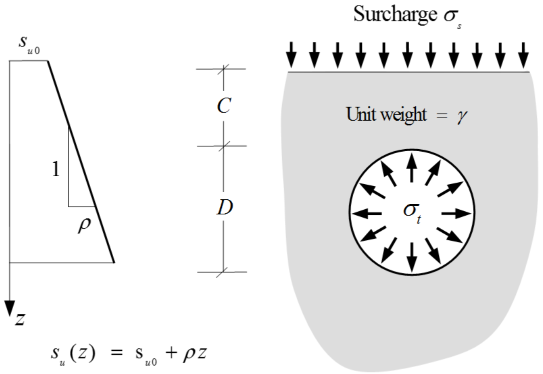


Figure 6. Measured and interpreted YSR profile (left) and permeability (right) vs range of values adopted by predictors.

Figure 7 shows a typical stability problem in geomechanics: a long straight tunnel of diameter D and cover C is constructed in a soil with a surface undrained shear strength s_{u0} and a linear strength gradient with depth of ρ. The tunnel is supported by an internal pressure σ_t with collapse being driven by the action of the surcharge σ_s and the soil unit weight γ. In practice, it is convenient to describe the failure of the tunnel by the stability parameter:

$$(\sigma_s-\sigma_t)/s_{u0} = f(C/D, \gamma D/s_{u0}, \rho D/s_{u0}) \quad (1)$$

where $\gamma D/s_{u0}$ and $\rho D/s_{u0}$ are, respectively, known dimensionless weight parameters and strength parameters, C/D is also known, and the number $(\sigma_s-\sigma_t)/s_{u0}$ needs to be determined. Figure 8 shows upper and lower bounds from FELA of the stability parameter $(\sigma_s-\sigma_t)/s_{u0}$, plotted as a function of $\gamma D/s_{u0}$ and $\rho D/s_{u0}$, for a tunnel with a cover-to-diameter ratio of C/D=4. These bounds bracket the exact stability parameter to within a few per cent and were found from adaptive finite element limit analysis using a maximum of around 4,000 elements. These types of stability charts can be used by tunnelling engineers to quickly obtain estimates of the internal tunnel support pressure that is required to prevent collapse.



Stability number

$$(\sigma_s - \sigma_t)/s_{u0} = f\left(C/D,\ \gamma D/s_{u0},\ \rho D/s_{u0}\right)$$

Figure 7. Schematic of the tunnel stability problem.

As well as providing the upper and lower bounds on the limit load, FELA also provides detailed information on the failure mechanism. Figure 9 shows such information for the case of a relatively shallow tunnel with *C/D*=4, $\gamma D/s_{u0}$=3 and $\rho D/s_{u0}$=1.

The left hand plot shows contours of the rate of plastic energy dissipation at failure, while the right hand side plot shows the corresponding velocity vectors, from which the plastic collapse mechanism can be inferred.

4 ROCK FRAGMENTATION AND PROTECTIVES STRUCTURES

Fragmentation of rocks during rockfall is a very complex phenomenon to understand and predict, as it depends on the mechanical and geometrical properties of both the falling rock and the impacted slope, as well as the block 2015) or structures that cannot adequately intercept dangerous fragments because that are inadequately located or not high enough.

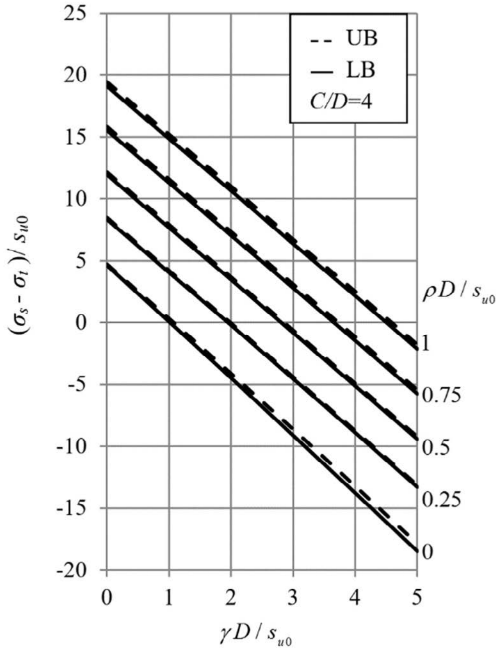


Figure 8. Stability bounds for a tunnel with C/D=4.

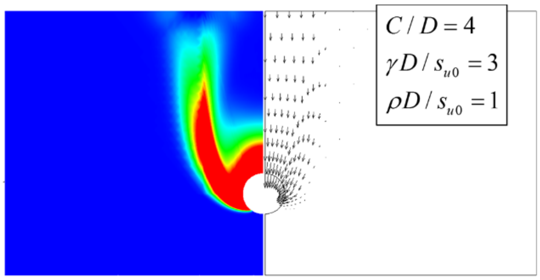


Figure 9. Failure mechanism for a shallow tunnel.

trajectory before impact. Knowledge and data on rock fragmentation during rockfall is scarce, despite its crucial importance for the design of protective structures. Not accurately accounting for fragmentation can lead to overdesigned structures (in terms of nominal energy), risk of mesh perforation (as per the bullet effect, Buzzi et al. CGSE researchers are investigating the mechanics of rockfall fragmentation with sophisticated 3D trajectory reconstruction (Fig. 10) and advanced instrumentation. This underpins the development of rockfall trajectory models that can predict occurrence and the outcome of fragmentation. This research is currently ongoing and will have an impact on the current practice of rockfall modelling.

The occurrence of rockfall in fractured rock masses is hard to prevent, so it becomes critical to adequately design protective structures. The past decade has seen the emergence of drapery systems as an alternative to rockfall barriers. Recent large scale in situ tests and Discrete Element (DEM) modelling by CGSE researchers (see Fig. 11) has led to a better understanding of the dynamic performance of drapery systems and a quantification of the residual risk posed by falling blocks that allowed the mining industry to better design entry portals. CGSE researchers have also contributed to advancing knowledge on low energy rockfall barriers (Buzzi et al. 2013) and the use of muckpiles or damping modules (Effeindzourou et al. 2017a, 2017b) as energy dissipation solutions. Cutting-edge research on drapery systems and attenuators is continuing, in collaboration with rockfall engineering company Geobrugg A.G., with the objective to optimise the design of such systems. This outcomes of this research are directly applicable to the design of road and rail cuttings, and work is now underway on this.

Figure 10. Post impact tracking of fragment trajectories (green lines) from high speed photographs (500 frames per second).

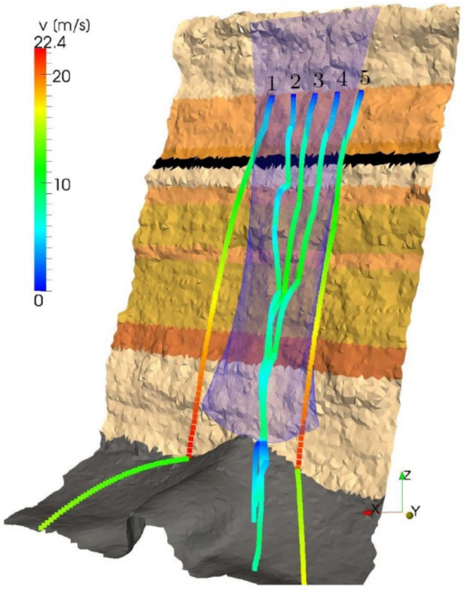


Figure 11. DEM modelling of trajectories of blocks falling under a drapery mesh installed on a high wall to protect a portal from rockfall impact (Thoeni et al. 2014).

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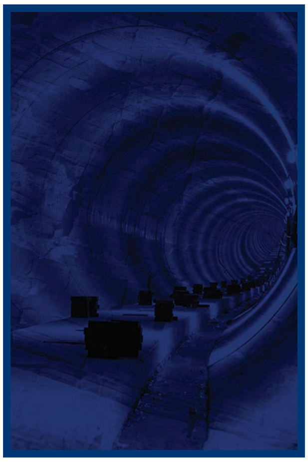

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