MEETING THE CHALLENGES OF COMPLEX EXCAVATION INTERACTIONS

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ABSTRACT

Design of buildings on and around existing underground infrastructure is becoming more and more necessary as land within the Sydney CBD and surrounds becomes a greater commodity and below ground space is being further utilised. In mining, many underground mines need to go deeper to be economically feasible, experiencing higher stresses and challenging conditions than ever experienced before in addition to complex geological settings with significant three-dimensional effects and multiple mining fronts. As a result, such complex and dynamic environment poses significant geomechanical challenges for the planning and design of such projects. The successful design of such projects is therefore fully dependent on a good understanding of what generates the complexity and the consequent impacts. Forecasting and predictive analyses are typically not needed for investigating such complexity and targeting cost-effective, sustainable and resilient solutions. Such analyses often involve large scale and complex 3D models that should be combined with experience based design and understanding of the fundamentals. This paper presents some discussions on how to address complexity with 3D modelling and present some modelling techniques that are useful to achieve reasonable results. Examples of model confirmation are also given to illustrate how some degree of confidence is gained based on available monitoring data and/or observations combined with local experience.

1 INTRODUCTION

One of the main challenges of the construction and mining industries is the continued and perhaps exponential increase in the complexity of the projects. Mills (2001) describes the construction industry as one of the most dynamic, risky and challenging businesses but one that has a poor reputation for managing risk, with many major projects failing to meet deadlines and cost targets. Mulholland and Christian (1999) add that construction projects are typically initiated in complex and dynamic environments that result in circumstances of high uncertainty and risk which are compounded by demanding time constraints.

Complexity can be difficult to define as it may have a number of different connotations. In particular, the word complex is sometimes used where complicated is meant. The Collins English Dictionary defines complexity as “the state or quality of being intricate or complex” with the definition of complex being “made up of many interconnecting parts”. As a result, complex should be used when dealing with something that consists of several parts rather than difficult to understand, analyse or deal with, which is what complicated inherently means.

In construction projects, complexity may come in a number of forms, for instance, large number of stakeholders, project subdivision into parts that are subcontracted as individual enterprises, multiple environmental constraints etc. All of these factors are essential for risk management from a global viewpoint.

From a geotechnical viewpoint alone, complexity quite often is a result of significant three-dimensional effects both in terms of geometry and geology and significant interaction between multiple excavations or components in the ground such as foundation or retention elements. For example, design of buildings on and around existing underground infrastructure is becoming more and more necessary as land within the Sydney CBD and surrounds becomes a greater commodity and below ground space is being further utilised. In mining, many underground mines need to go deeper to be economically feasible, experiencing higher stresses and challenging conditions than ever experienced before, and typically involving excavating in complex geological settings with significant interaction between multiple mining fronts. Open pit mining has more often involved excavations near sensitive, environmentally or heritage protected structures

Such a complex and dynamic environment pose a significant challenge for the planning and design of these projects. The successful design of such structures is fully dependent on a good understanding of the potential interactions between the interacting “parts” and consequent impacts. As a result, forecasting and predictive analyses provide the necessary tools to make such assessments. These analyses will inherently involve large scale and complex 3D models that should be combined with experience based design and understanding of the behaviour of the “individual parts” of the model but also of the fundamentals of geotechnical behaviour.

An important step of such analyses is good model confirmation due to the degree of complexity associated with the input data assumptions (e.g. geological model, material parameters and models, initial stress condition, construction sequence etc.).

This paper presents some discussions on how to address geotechnical complexity with advanced 3D modelling and present some modelling techniques that are useful to achieve reasonable results, for example sub-modelling and assembly of multiple parts. Examples of model confirmation are also given to illustrate how some degree of confidence is gained based on available monitoring data and/or observations combined with local experience. The role of material behaviour and the importance of focusing on relevant behaviours and mechanisms are also briefly discussed.

2 MODEL CONFIRMATION AND CALIBRATION

There is a general notion that numerical simulation models need to be "verified" and/or "validated". These two terms are often used erroneously and interchangeably although they have different meanings. The author of this paper often falls in such a trap although he agrees with the following discussion provided in Oreskes et al (1994).

The word "verify" comes from the Latin "verus" which means true. Therefore, "verify" means an assertion or establishment of "truth". To say that a model is verified implies that its truth has been demonstrated which also implies its reliability as a basis for decision-making. However, this is only valid for closed systems with unique answers.

Natural systems such as geotechnical problems, on the other hand, are never closed and model results are always non-unique. One reason they are never closed is that these models require input parameters that are incompletely known. Another problem arises from the scaling-up of non-additive properties. The construction of a numerical simulation model of a geotechnical problem involves the specification of input parameters at some chosen scale. Typically, the scale of the model elements is in the order of meters, tens of meters, or kilometres. In contrast, the scale on which input parameters are measured is typically much smaller, and the relation between those measurements and larger scale model parameter is always uncertain and generally unknown. In some cases, it is possible to obtain input data at the scale chosen by the modeller with large scale field tests but this is not often done, for practical reasons. Even when such measurements are available, they are never available for all model elements. In summary, it means that it is impossible to demonstrate the truth of an open system.

Validation typically denotes the establishment of model legitimacy in terms of arguments and methods, and not necessarily the truth, although truth is not precluded. For example, a valid contract is one that has not been nullified by action or inaction. A valid argument is one that does not contain obvious errors of logic. By analogy, a model that does not contain known or detectable flaws and is internally consistent can be said to be valid. However, the predictions of such a model may or may not be valid, depending on the quality and quantity of the input parameters and the accuracy of the auxiliary hypotheses.

Despite the fundamental problems of model verification, engineers and modellers still need some confidence in model predictions and their reliability as a basis for decision-making, which may lead us to a simple "model confirmation".

Models can be confirmed by the demonstration of agreement between observation and prediction, but confirmation is inherently partial. The greater the number and diversity of confirming observations, the more probable it is that the conceptualization embodied in the model is not flawed. In other words, confirming observations do not demonstrate the veracity of a model or hypothesis, but they support its probability of success.

Calibrations may form part of the model confirmation process. One could say that a calibrated model is empirically adequate with the argument that the goal of any scientific theory is not the truth (unobtainable) but empirical adequacy. However, it is also important to note that even if a model result is consistent with present and past observational data as a result of such calibration, there is no guarantee that the model will perform at an equal level when used for forecasting predictions. There may be small errors in input data that do not impact the fit of the model under the assumed conditions, but which, when extrapolated, may generate significant deviations. Second, a match between model results and present observations is no guarantee that future conditions will be similar because natural systems are dynamic and may change in unanticipated ways.

Nevertheless, model confirmation and calibration is an essential step in any numerical modelling work due to the degree of complexity associated with the input data assumptions (e.g. geological model, material parameters and constitutive models, initial stress condition, construction sequence etc.). It should typically involve a comparison of the results with other methods or previous experiences to demonstrate that there is a reasonable representation of the anticipated behaviour over a particular range of conditions under consideration, therefore with probable success of future predictions.

In geotechnical engineering, the analysis results are often compared to local past experience where similar construction methods were adopted in similar ground conditions, or with even higher degree of confidence when construction monitoring data is available from the same project.

2.1 EXAMPLE OF MODEL CONFIRMATION FOR MULTIPLE PARTS

This section presents a case study of a property redevelopment in Sydney on top an existing rail station. The example seeks to illustrate the process of model confirmation of a complex problem, i.e. with a number of interacting components, where the predictions of some of the individual parts are compared to past observations and/or local experience. As discussed above, the model confirmation is not meant to verify the "truth" of the future predictions but to increase the probability of success.

The proposed redevelopment comprises the reconfiguration of an 18 level building with a 3 level podium occupying the total site footprint and a 15 level tower above this. A rail station was built in the 1970's immediately under building and settlement of the building and the pavement immediately above the station concourse was monitored during construction. Some of the building footings are immediately over the platform access tunnels (escalators) and others immediately on the side walls of the concourse. There is a complex level of interaction between the multiple parts (several foundation components and tunnels) with significant 3D effects. The overall geometry/lay-out of the redevelopment is shown in Figure 1.

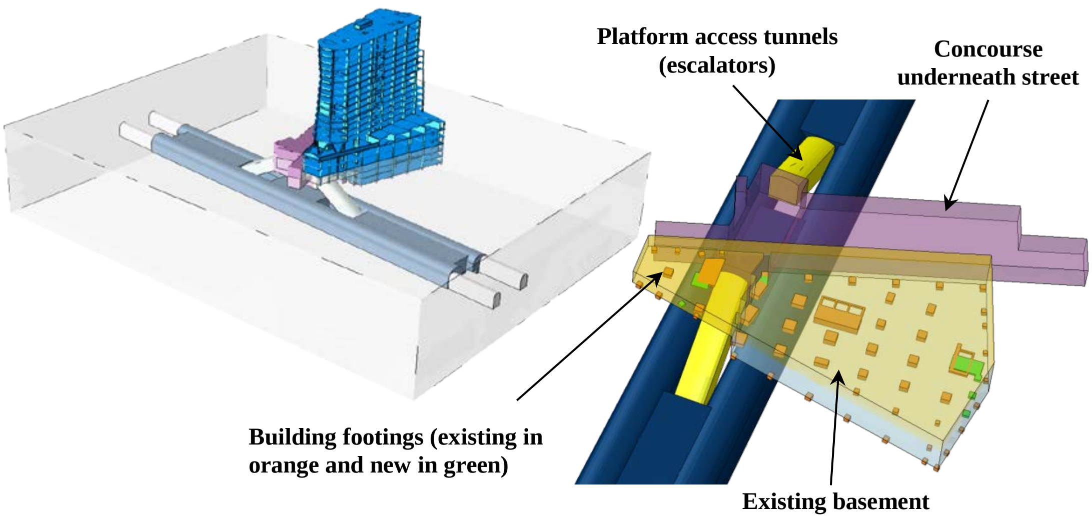

Figure 1: Complete 3D model of example 1 with detail close-up on the right hand side.

The reconfiguration involves a new building façade with minor extension of the tower floor area and some minor load increase on some of the existing footings. The extension of the tower is located at the end of the building over the rail easement and new footings are required to support it. In addition, alteration/reorientation of the building lift core over the rail easement is also required with load changes in the vicinity of the rail tunnels. The existing basement will be retained and no further bulk excavations are proposed.

As part of the approval process it is required to demonstrate that the proposed redevelopment induces minimum impact on the rail asset. In order to do so, the current condition of the structure of the station, including existing loads on tunnel liners, concourse roof and walls etc., needs to be assessed. This involves trying to represent the construction of the station and excavation of the tunnels immediately under the existing building.

The geology consists of Hawkesbury Sandstone with quality improving with depth, from Class V near surface with thickness of up to 8 m to Class II Sandstone in the platform tunnels. It is well known that the excavation of such tunnels will not only be dependent on the rock mass behaviour and associated parameters but also the initial stress condition which is another major uncertainty.

2.2.1 Existing basement excavation

Published data, including Sydney experience, suggests that lateral movements of an adequately designed excavation and/or retention system in hard clays and/or weathered rock will typically be between 0.1% and 0.2% of the excavation depth

Vertical excavations in good quality shale and sandstone will also result in lateral movement of the rock faces into the excavation, mainly associated with in-situ stress relief (locked-in horizontal tectonic stresses). Previous experience and published data indicates that lateral movements in Sydney rock typically range between about 0.5 mm to 2 mm per metre depth of rock excavation. The extent of the horizontal movement behind the excavation face may vary between 1.5 and 3 times the excavated height with diminishing effect away from the face.

The amount and timing of movement will be dependent on the depth of excavation, rock quality, stress condition and the location and condition of bedding defects (such as weak bands or bedding partings). Modifications to shoring systems such as increasing anchor loads above those typically adopted may result in some reduction of movement. However, the extent of the reduction will be highly dependent on the excavation and construction sequence and it may be difficult in practice to achieve significant reductions in movements due to the relatively high in-situ stresses.

The above experience provides useful information that can be used for model confirmation purposes although no measurement of the existing building excavation is available. Figure 2 provides an overview of the lateral movements predicted in a 3D model of the case study as a result of the existing basement excavations alone. A summary of the lateral movements predicted in the model as a result of the deep excavations is also presented with a comparison with the typical range of lateral movements often observed in Sydney (Oliveira and Wong, 2012). The results indicate satisfactory model predictions ranging between 0.05% and 0.1% of the retained height which is consistent with local experience.

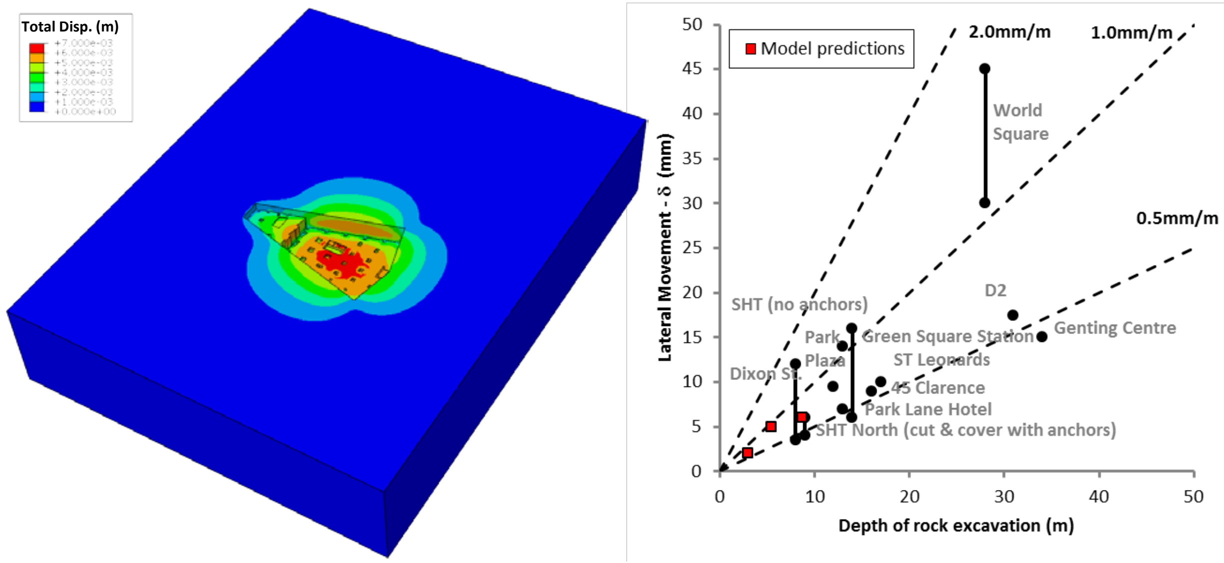

Figure 2: Comparison of predicted lateral displacements of deep basement excavations with previous experience (modified after Oliveira and Wong, 2012).

2.2.2 Platform cavern excavation

Settlement monitoring was carried out during excavation of the platform tunnels providing very useful data for model confirmation.

Excavation of the platform caverns resulted in monitored maximum surface displacements in the order of 9 to 10 mm. Figure 3 shows the model prediction at the end of the platform excavation with the approximate location of the section monitored between 1969 and 1970. Figure 4 compares the surface settlement trough measured during excavation with that predicted with the current model. This figure indicates reasonable agreement with some minor differences likely associated with the stiffening effect of the building structure not captured in the current model.

2.3 ROLE OF MATERIAL BEHAVIOUR AND IN-SITU STRESS IN MODEL CALIBRATION

As discussed above, model calibration may often form part of the confirmation process. However, the admission that calibrated models invariably need "additional refinements" suggests that the empirical adequacy of such a numerical model is inherently forced. This need for “refinement” may have serious consequences on future predictions as the behaviour calibrated for a single observation may not be representative of others. This is particularly relevant if the underlying assumptions of material behaviour and initial conditions such as stress do not adequately capture the appropriate mechanisms involved in the problem being analysed.

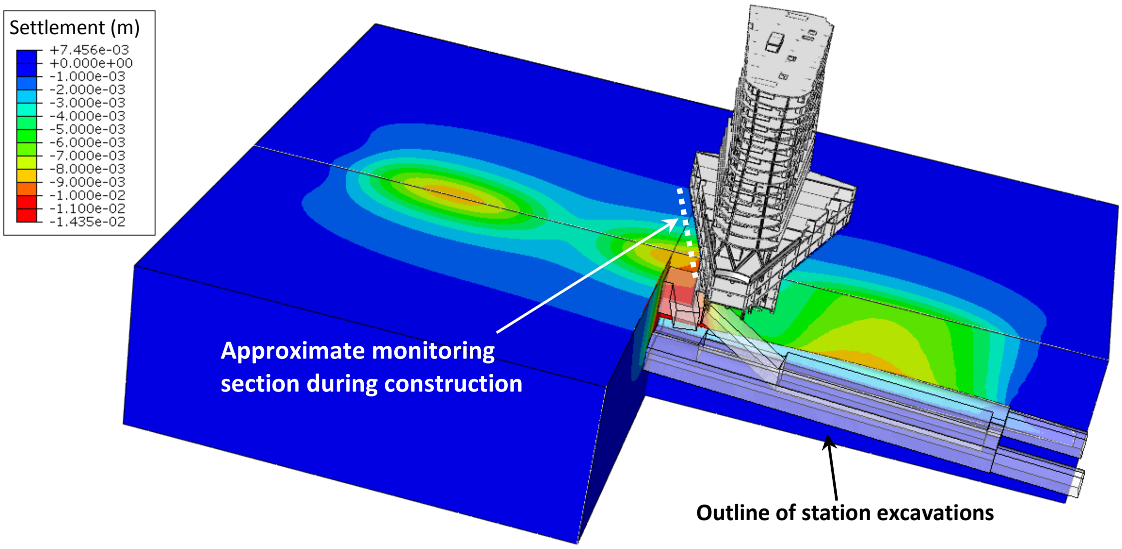

Figure 3: Cut-away of induced settlement at the end of platform cavern excavation.

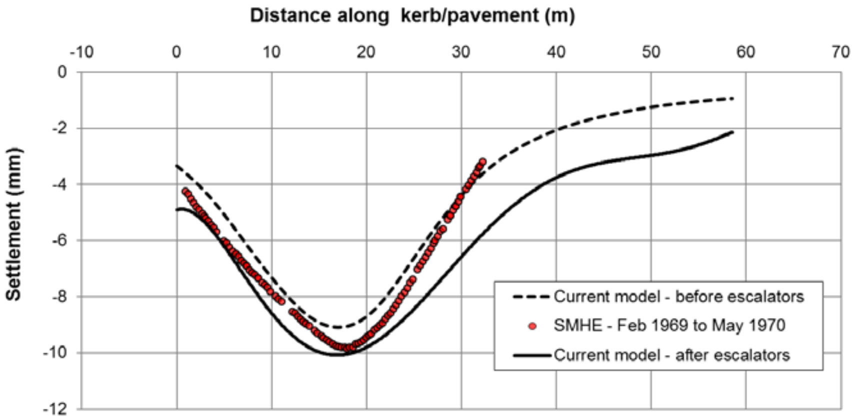

Figure 4: Comparison between model predictions and values observed during cavern excavation.

In the assessment of complex interaction mechanisms where a potential impact is investigated, material deformation and initial in-situ stress conditions are typically the governing factors, significantly more than failure criteria and strength parameters.

In Sydney Hawkesbury Sandstone it is well known that the typical bedding spacing is 1 to 5 m with the two orthogonal sub-vertical joints (NNE and ESE) presenting much wider spacing varying from 2 to 20 m (Bertuzzi and Pells, 2002). This alone indicates that such horizontally bedded sedimentary rock is in many cases transversely isotropic material both by way of the preferred orientation of particles, bedding textures and bedding partings, in addition to other effects related to discontinuity infillings (clay seams) and interlaminations. Such anisotropy induces equivalent rock mass elastic properties in the horizontal direction that are generally higher than that in the vertical direction. Another important aspect of such transverse isotropic behaviour is that the shear modulus G_{vh} is also significantly affected by the discontinuities and needs to be corrected independently since it does not follow the classical isotropic relationship with Young's Modulus and Poisson's ratio. Further discussions can be found in Oliveira and Wong (2012) and Oliveira (2014).

It is also known that increasing confining stresses also promotes an increase in rock mass modulus (Figure 5). Therefore, such anisotropic effect of the Hawkesbury Sandstone is further enhanced by the fact that the horizontal stresses in Sydney are generally higher than the vertical. In other words, the higher confinement acting on the orthogonal joints compared with those acting on bedding partings will induce values of horizontal modulus higher than the vertical.

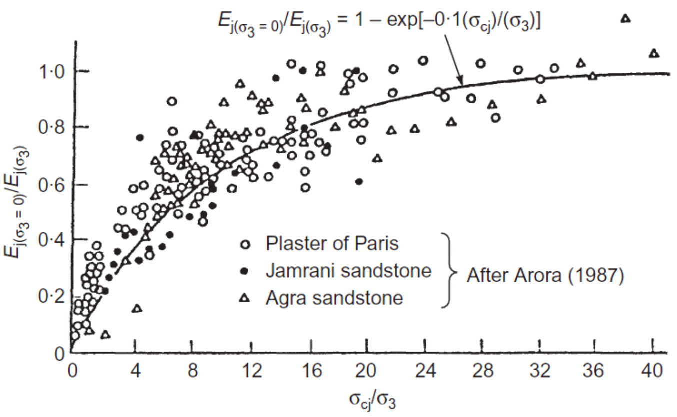

Figure 5: Variation of $E_{j(\sigma3=0)}/E_{j(\sigma3)}$ for jointed rocks (after Ramamurthy, 1995).

Despite the above, the author of this paper rarely sees the application of different modulus in the horizontal and vertical. Model calibration based on isotropic elasticity would likely result in limited rock mass behaviour representation where only a particular mechanism would be represented. For example, a calibration based on settlements (i.e. vertical movements) would less likely satisfy the prediction of lateral movements. The exception would be when discontiuum models are used where rock discontinuities are explicitly accounted for anisotropy is inherited from the different rock defect spacing adopted in the model.

Another uncertainty and factor of significant influence is the initial in-situ stress condition. In-situ stresses will certainly influence the predicted behaviour with respect to movements and potential interaction between multiple components.

A number of studies (Oliveira and Parker, 2014) have indicated that large horizontal stresses are locked into the sandstone rock in Sydney. The major principal stress, σ_1, is approximately horizontal with a typical trend of 0 to 20º from the true north (i.e. NNE). The intermediate stress, σ_2, is also horizontal and the minor principal stress, σ_3, is equal to the vertical stress, both orthogonal to the major principal stress. As expected, the major principal stress is approximately oriented in the same direction of some of the major fault zones observed in Sydney.

Despite the available studies in Sydney, the decision on what initial stress condition to adopt in modelling is not a straight-forward task. However, Oliveira and Parker (2014) presented a database of stress measurements in the Sydney Basin and proposed that the decision should be based on rock mass quality, and consequently its stiffness.

2.3.1 Example of model confirmation of rock mass anisotropy and initial in-situ stress conditions

This section presents a case study of a property redevelopment involving basement excavation and foundations in the vicinity of the North Ryde Station (Figure 6). The example seeks to illustrate the process of confirmation of adequate rock mass behaviour for the same model part but investigating different deformation mechanisms, i.e. vertical and lateral.

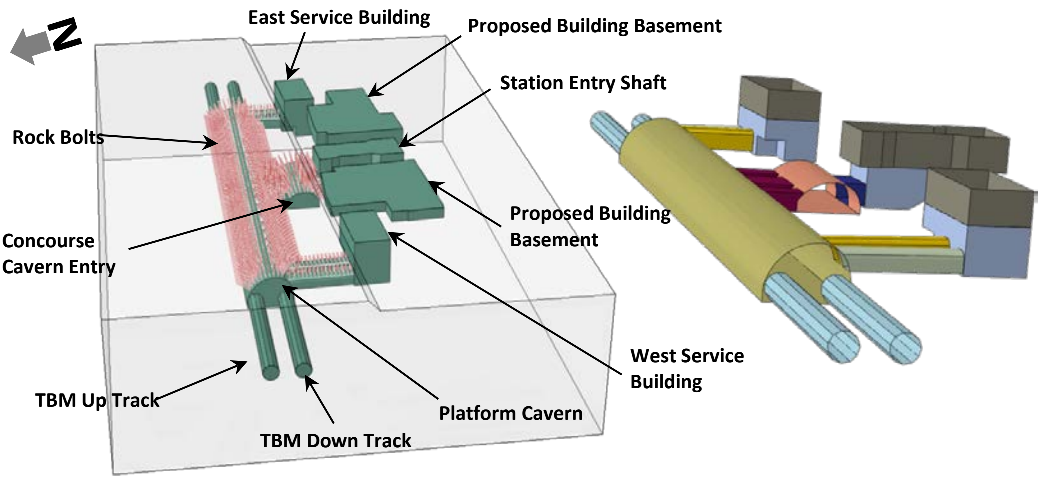

Figure 6: Complete 3D model of example 2 with detail of station structures on the right hand side.

As part of the Epping to Chatswood Rail Line (ECRL) design, Chan et al (2005) stated that "unusual" in-situ stress conditions were observed at North Ryde Station, i.e. high horizontal stresses, that could not be explained by topographic effects. These authors also stated that there was enough evidence to support these high stresses through the presence of stronger, less jointed sandstone and some crushing and localised shearing of bedding plane seams. Such high stress was also further validated during construction through back-analysis of monitoring data (Chan and Stone, 2005).

Although Chan et al (2005) used the term "unusual", the database presented in Oliveira and Parker (2014) indicates that such high stresses are also observed elsewhere in Sydney. Oliveira and Parker (2014) show that for better quality sandstone, the in-situ stress relationships are likely higher than the average stresses in Sydney and present stress relationships that are fully consistent with that adopted by Chan et al (2005). As a result, for numerical modelling purposes, it was considered appropriate to adopt the in-situ stress proposed by Chan et al (2005) as:

$\sigma_1 = \sigma_H = 1\ \text{MPa} + 6\sigma_v$

$\sigma_2 = \sigma_h = 0.7\ \text{MPa} + 4\sigma_v$

The above relationship was assumed applicable for the Class I/II Sandstone unit with in-situ stresses adjusted for the different rock qualities based on the approach proposed by Oliveira and Parker (2014).

As suggested by Chan and Stone (2005), the geotechnical monitoring data of the Macquarie Park Station caverns, available during construction, provided very useful data for calibration of the other three ECRL station caverns while they were still being designed, including the North Ryde Station. This is because it was built in very similar geological conditions and with very similar geometry to the others, for example the North Ryde Station. As a result, considering that data from the North Ryde Station was not available to the author, it was considered appropriate to compare some of the monitored performance of the Macquarie Park Station caverns excavation to the model predictions of the North Ryde Station.

Excavation of the Macquarie Park Station platform and concourse caverns top heading resulted in monitored surface displacements in the order of 9 to 13 mm (Chan and Stone, 2005). Figure 7 shows the model prediction at the end of the top heading excavation. Model prediction at a similar section as that back-analysed by Chan and Stone (2005) indicates surface settlement of approximately 12 mm consistent with the performance observed at the Macquarie Park Station.

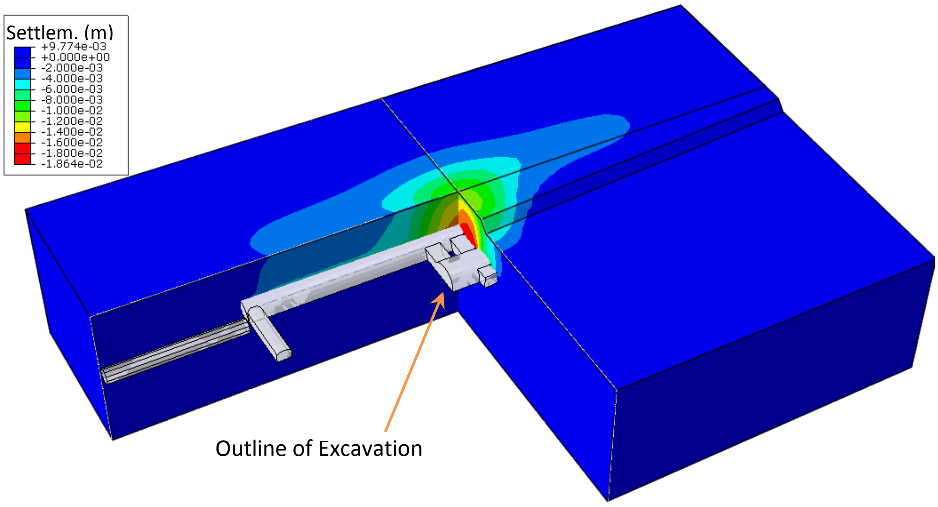

Figure 7: Cut-away of induced settlement at the end of platform and concourse caverns top heading excavation.

Chan and Stone (2005) also indicated that sidewall convergences in the order of 23 to 27 mm were observed upon completion of a full heading. Figure 8 shows model predictions with estimated wall convergence of 24 to 30 mm and are therefore consistent with similar observations at Macquarie Park Station.

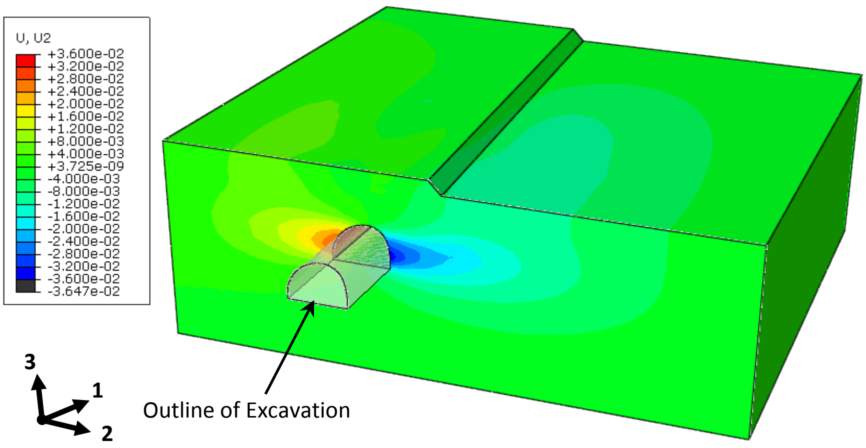

Figure 8: Section cut of induced horizontal displacements (U_2) upon completion of a full heading excavation

For the example above, it was considered appropriate to adopt geotechnical units and parameters somewhat similar to those presented in Chan et al (2005) and Chan and Stone (2005), as these were adopted for design purposes for the ECRL and back-calibrated based on monitoring data during constructing of the station caverns. However, an equivalent continuum model with different values of vertical and horizontal modulus was adopted.

The stiff residual soil, Rs, and Asc units were modelled as isotropic elastic – perfectly plastic material following a Mohr Coulomb failure criterion. The MFa, MFb and HAWa units were modelled as transversely isotropic elastic – perfectly plastic materials with failure through either the intact rock (matrix) or ubiquitous “joints” representing the three main discontinuity sets (bedding and two sub-vertical joint sets). The vertical components of the elastic modulus, E_v, are those typically adopted for foundation design in the Sydney area, which are basically values assessed under loading condition. The horizontal component of the elastic modulus, E_h, is assumed approximately 3 times the value of the vertical modulus based on the discussion above. The adopted parameters for the geotechnical units used in the numerical analysis are presented in Table 1. The parameters adopted for the ubiquitous “joint” sets are presented in Table 2.

Table 1: Parameters adopted for rock units.

Unit	σ_{ci} (MPa)	Rock mass ϕ (deg)	Rock mass *c* (MPa)	σ_{ti} (MPa)	E_v (MPa)	E_h (MPa)	G_{vh} (MPa)	Poisson ratio
Rs	-	25	0	0	60	60	22	0.35
ASc	1-2	28	0.02	0.05	115	115	45	0.3
MFb	10-15	35	0.5	0.25	400	1200	120	0.25
MFa	10-40	40	2	1.5	600	1800	180	0.25
HAWa	15-40	45	3	2	1000	3000	240	0.25

Table 2: Parameters adopted for discontinuities.

Joint	Dip Direction (deg)	Dip (deg)	ϕ^1 (deg)	*c* (kPa)	Dilation angle, *i* (deg)	σ_t (kPa)
Bedding	0	0	26 / 32	5	5	0
NNE	110	80	26 / 32	5	5	0
ESE	20	80	26 / 32	5	5	0

Notes: 1) First value corresponds to discontinuities within MFa and MFb and second within HAWa units.

3 SOME MODELLING TECHNIQUES

One of the challenges of modelling complex problems is that due to the number of multiple interacting parts the model can become relatively large. Such large 3D models will inherently demand high computation cost and power if reasonably good quality meshes are to be targeted.

Other engineering industries such as the automotive and aerospace industries have faced the similar issues and significant investment has been made to address such a problem. For example, during the past two decades, crashworthiness simulation of automotive structures has proven to be remarkably good, largely because the finite element codes being used can accurately predict the plastic bending and stretching deformation mechanisms that occur in stamped metal parts. Leading companies such as BMW has more and more often relied upon computer simulation to advance their designs which has pushed considerable advances in the numerical simulation industry which can be adopted for the modelling of geotechnical problems.

3.2 MULTIPLE PARTS MODELLING

Considering that in the automotive and aerospace industries the final product is always intrinsically made of an assembly of multiple parts where the connections are relevant for the performance and behaviour of the entire system, the assembly of parts are also translated to numerical models targeting a more realistic simulation and representation of the observed behaviour (Figure 9). The techniques used in such assemblies can also be used in geotechnical models to allow refined meshes were required without significantly increasing the computation cost demand.

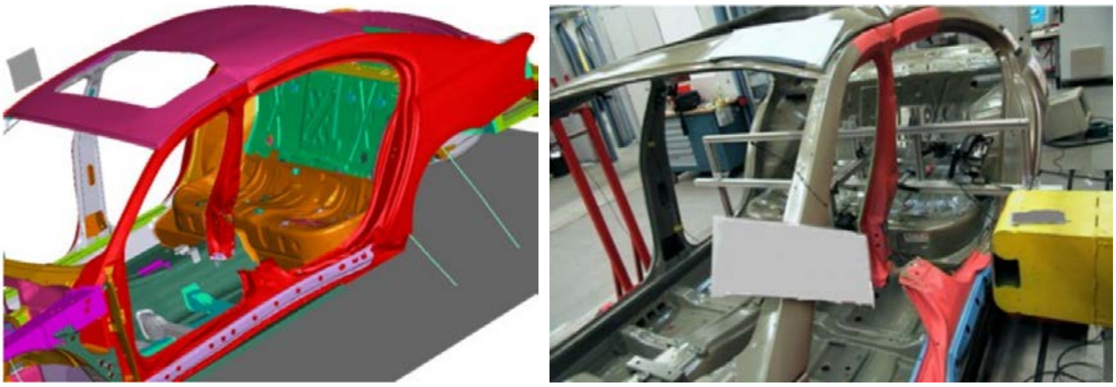

Figure 9: Example of complex automotive model involving multiple parts (Simulia, 2008).

Two primary techniques can be used to work with models of multiple parts:

- Constraints: such a technique partially or fully eliminates degrees of freedom of a group of nodes and couples their motion to the motion of a master node (or nodes). Examples of constraints include:
  - Rigid Bodies
  - Coupling Constraints
  - Shell-to-Solid Coupling
  - Surface-Based Ties
- Connectors: they model actual connections between parts with specific behaviours, such as: ball joints, springs, dampers, bushings, links, hinges, spot welds, rivets, adhesives, bonds, etc. Examples of finite element connections include:
  - Connector Elements
  - Mesh-Independent Point Fasteners
  - Mesh-Independent Surface Connections

Just as an example, Figure 10 presents the technique adopted in example 1 of this paper. Considering that the primary objective of the analyses was to assess the potential impact of the proposed redevelopment on the structures of the rail station, it was considered important to have a structural mesh that was fine enough to appropriately capture the potential bending induced by the foundation loads. The level of detail of the structure, including localised concrete columns, would impose a very large mesh if fully combined with the actual ground model. As a result, the structural mesh was generated completely independent of the ground mesh and “tied” to the ground model.

A “node-to-surface” approach is often used in these cases. The computer code used for the cases presented in the paper, Abaqus/FEA from Dassault Systemes, uses a position tolerance criterion to determine the constrained nodes based on the distance between the slave nodes and the master surface. Abaqus sets tie interpolation coefficients equal to the interpolation functions at the point where the slave node projects onto a master surface. For the node-to-surface method of establishing the tie coefficients with an element-based master surface, the point on the surface closest to each slave node is calculated and used to determine the master nodes that are going to form the constraint. For example, in Figure 10 nodes 202, 203, 302, and 303 are used to constrain node *a*; nodes 204 and 304 are used to constrain node *b*; and node 402 is used to constrain node *c*.

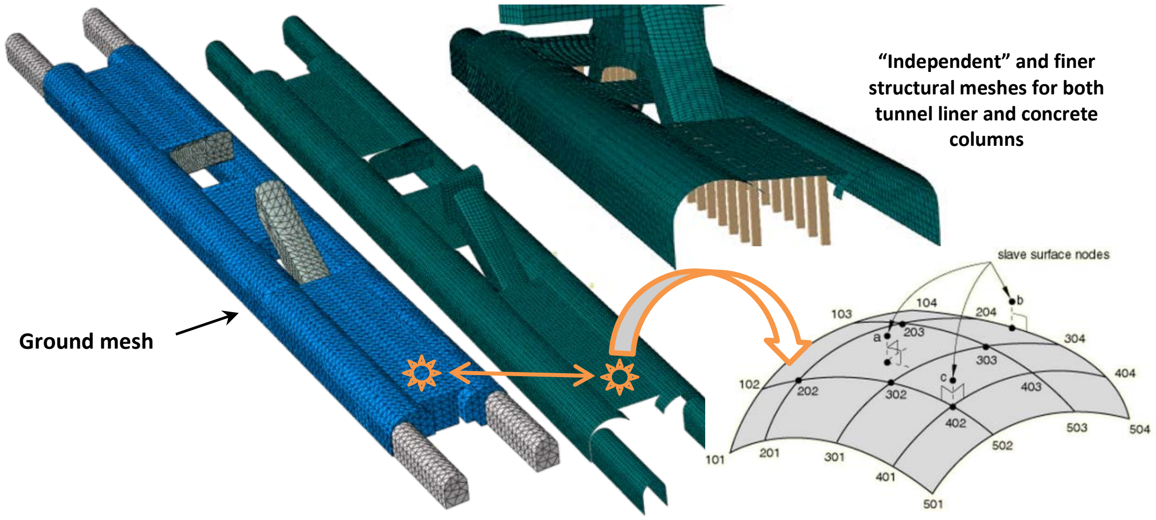

Figure 10: Independent finer structural meshes "tied" to global model mesh and each other in example 1.

3.3 SUB-MODELLING

In certain cases, the main challenge is not the use of multiple parts but the scale of the problem and level of details which may lead to very large models if attempts are made to include everything in a single 3D model.

For example, deep mining often involves excavating in complex geological settings with significant three-dimensional effects both in terms of geometry and geological structure such as faults, shears and folds. In addition, there is often significant interaction between multiple mining fronts and between stopes and key infra-structure such as development drives, shafts, raises and caverns. As a result, such complex and dynamic environment poses a significant modelling challenge, particularly from a scale perspective.

To account for all this complexity, mine-scale models are often required. On the other hand, it is still necessary to impose practical limits to the size of such models due to not only the required computer power but also to obtain results within reasonable time frames.

The result of such model limitation is that the meshes of mine-scale models are typically not fine enough to capture the appropriate plasticity and rock damage at a smaller scale, for example, to investigate damage around development drives and ground support efficiency. As a result, smaller scale models become necessary.

To maintain a reasonable mesh size in detailed smaller scale models featuring a finer mesh, they typically do not include all the multiple excavation fronts or 3D geometrical or geological effects that are outside the model boundaries and that may affect the stress redistribution around the excavations. As an attempt to compensate for this limitation, the geomechanics mechanisms addressed with such smaller scale models are often simplified, targeting results that are thought to be conservative but that may not always fully represent the actual mine behaviour.

Sub-modelling technique allows for a multi-scale simulation approach. In such an approach, a three-dimensional mine-scale or global model is used to model the entire mine behaviour through its entire Life of Mine and sub-models, with finer meshes, used to target specific areas. The boundaries of the sub-models are controlled by the mine-scale model to ensure compatibility of displacements and stress between the different scales.

Figure 11 presents an example described in details in Oliveira et al (2014). A 3D finite element model of the entire mine was constructed for analysis but only capturing the major extraction volumes (stopes) that would induce significant changes in stress condition. A sub-model, shown in the close-up of Figure 11, was then constructed to focus on future production areas for particular mining levels. It contained greater detail than the global model. First, both development and sill pillars were included in the sub-model. Second, generally smaller high order element sizes were used, allowing both better description of drive and stope geometry, and better resolution of stress and damage results adjacent to excavations. The results were then used to assess and identify areas of higher mining risk and provide information for geotechnical engineering assessment, including the interaction between different mining fronts.

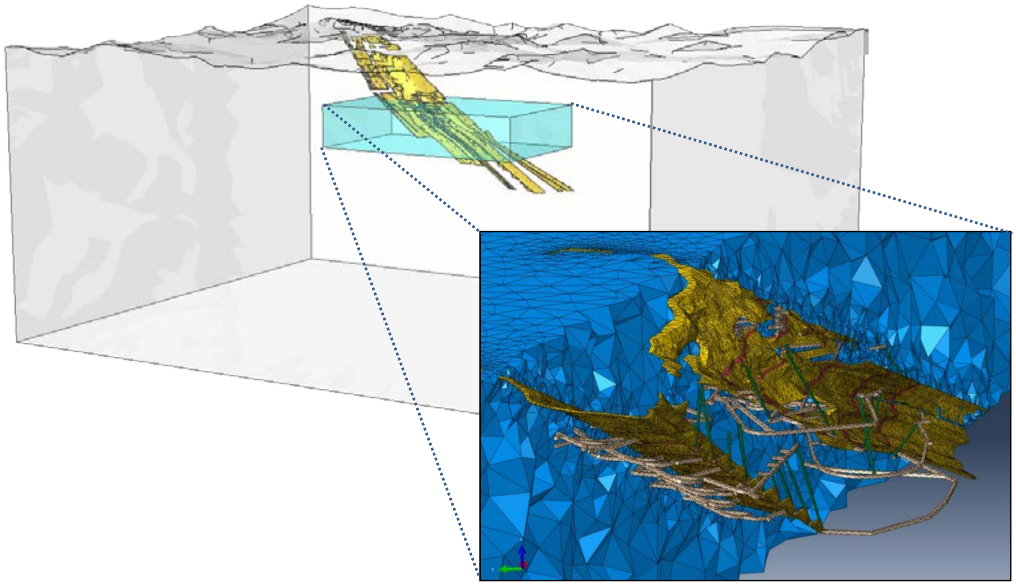

Figure 11: Example 3 illustrating sub-modelling technique for a deep undeground mine.

An important aspect of a sub-modelling approach adopted is that the boundaries of the sub-models are controlled by the previous mine-scale model to promote compatibility of displacements and stress between the different scales. In other words, the effects of mining extractions that occur outside the sub-model region are accounted for through the boundary displacements controlled by the global scale model. The location and bounding boundary box of the example 3 is presented in Figure 11. Creating a sub-model is a two-step process. First a global model needs to be analysed. The sub-model is then created with boundary conditions which are controlled with time-dependent variables that were saved during the analysis of the global model. Sub-model boundaries can be controlled by either displacement conditions or in some cases stresses from the global model.

4 CONCLUSIONS

The paper presented some discussions on how to address geotechnical complexity with advanced 3D modelling including some brief discussions on modelling techniques that are useful to achieve reasonable results, for example sub-modelling and assembly of multiple parts. Examples of model confirmation were given to illustrate how some degree of confidence can be gained based on available monitoring data and/or observations combined with local experience. The role of material behaviour and the importance of focusing on relevant mechanisms were also briefly discussed.

The use of such advanced techniques can provide invaluable information for decision-making and successful design of projects where significant and complex interaction may be present.

Despite the discussion in the paper, it is important to note that models should only be evaluated in relative terms, and their predictive value is always open to question. The primary value of models is heuristic, i.e. to be used to corroborate hypotheses by offering evidence to strengthen what may be already partly established through other means. Models can elucidate discrepancies in other models and also be used for sensitivity analysis exploring "what if" questions thereby illuminating which aspects of the system are most in need of further study, and where more empirical data are most needed. The later aspect highlights the importance of geotechnical monitoring through any proposed project.

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