TEMPORARY SUPPORT OF DEEP BASEMENT EXCAVATIONS IN ROCK

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ABSTRACT

Deep basements have become common in our modern cities. Whilst the analysis and design of retention systems for deep basements in soil are relatively well established, the same cannot be said for deep basement retention systems in rock. In many instances the design of retention systems in rock are based on a soil mechanics approach and as a result often ignore unique aspects of rock mass behaviour that can significantly impact the performance of these retention systems. It is important that the characteristics of the rock mass are well understood and are quantified during the ground investigation. Some of the aspects of retention system design and analysis that are set out in this paper only became apparent following unsatisfactory performance of a retention system. Due to the sensitivity surrounding such unsatisfactory performance, it has not been possible to include the specific case studies. As a result, this paper provides an overview of the many aspects involved in respect to ground investigations, analysis, design and construction of deep basement retention systems in rock that are required to mitigate against unsatisfactory performance. This paper is restricted to temporary embedded wall retention systems that are installed to allow the construction of the permanent basement structure.

1 INTRODUCTION

Deep basements have become common in our modern cities. Height restrictions, local ordinances and building amenity often necessitate that new multilevel buildings include up to several levels of below ground basements for parking.

Basement construction is commonly undertaken by installing piles, diaphragm walls or similar around the edges of the planned excavation prior to the excavation commencing. As the excavation proceeds, rows of post tensioned ground anchors (or in some cases soil nails/rockbolts) or struts are installed to provide support to the piles/diaphragm wall. The ground anchors/struts are the main structural elements that provide temporary support to the piles/diaphragm walls to resist the earth pressures that develop as a result of removal of the ground from the excavation. Once the excavation is completed, the anchors and struts are usually replaced by basement floor slabs which form part of the permanent basement structure. The piles/diaphragm walls may be incorporated into the permanent structure or may be supplemented by permanent structural walls.

For most competent builders, the construction of the temporary basement retention system (hence forth referred to as retention system) is considered a high risk activity and is usually on the critical path for construction. Issues encountered during construction of the retention system can cause significant damage and construction delays and pose a risk to workers and the public. It is therefore paramount that the analysis, design and construction of the retention system are carried out competently and prudently with a full understanding of the ground related hazards that may be present.

Undertaking a basement excavation modifies a site from a usually stable situation into a potentially unstable one. The removal of the ground to form the excavation removes the support to the adjoining land and provides a space for which the adjoining ground can move or collapse into. Although not considered in this paper, the excavation can also locally change the groundwater regime which can also cause significant issues.

To limit movements and prevent collapse of a deep vertically sided excavation a retention system is required. The primary safety requirement of a retention system is that it prevents the walls of the excavation from collapsing. However, also of prime importance is that the retention system limits movement of the excavation walls and the surrounding land. This movement occurs as a direct result of the reduction of in situ stresses in the ground due to the excavation. Whilst the change in situ horizontal stresses is usually of most concern, as discussed below the reduction in in situ vertical stress can also be of significance. In general, the stiffer and more robust the retention system, the lower will be the reduction in in situ horizontal stress and hence the lower the ground movement due to the excavation.

A difficult design decision is often associated with what magnitude of movement is reasonable, and what is the lowest movement that is practically and reasonably achievable. This has a direct impact on the stiffness, robustness and cost of the retention system. A retention system that is too "light" could potentially result in unsatisfactory movement and damage to adjoining assets and hence significant cost implications with possible significant delays during construction. One that is too "heavy" has significant cost and program implications. The aim is to achieve a retention system that strikes a balance between these two extremes but includes a reasonable level of prudency.

Whilst the analysis and design of retention systems for deep basements in soil are relatively well established, the same cannot be said for retention systems in rock. For this reason, many designers use a soil mechanics approach to analyse and design retention systems in rock and as a result often ignore unique aspects of rock mass behaviour that can significantly impact the performance of a retention system in rock. It is important that the characteristics of the rock mass are well understood and are quantified during the ground investigation. This paper sets out the authors' experience in respect to ground investigations, analysis, design and construction of deep basement retention systems in rock.

2 CRITICAL GROUND PROPERTIES FOR DESIGN

The primary aim of any temporary retention system is that it must be able to adequately resist the horizontal pressures which act or will act on the retention system as a result of removal of the ground within the excavation. These horizontal pressures include the in situ horizontal stress within the ground, ground water pressure and pressures due to nearby surcharges. Earthquake loads need to be considered for the permanent basement structure, but not usually for the temporary works.

The maximum change in in situ horizontal stress that can occur for a stable retention system is from the at rest state, K_0, to the active pressure state, K_a. It follows that in order to prevent collapse, the retention system must, as a minimum, be strong enough to resist the active (K_a) earth pressure as well as any other applied loads (due to groundwater and surcharges). The strength of the ground (which defines K_a), the groundwater conditions and applied surcharges are therefore key parameters in the design of a retention system.

However, a further key requirement is that a retention system also limit ground movements to tolerable levels. In the simplest case where there are no groundwater pressures or surcharges, horizontal movements occur if the horizontal stresses imposed by the retention system to the walls of the excavation are either less than or greater than K_0, with greater movements occurring the greater deviation from K_0. Vertical movements of the adjacent ground primarily occur as a result of the horizontal movements (resulting in settlement) but to a lesser extent also from the removal of vertical stress from within the excavation (resulting in heave).

Movements of the retention system walls towards the excavation are accompanied by a reduction in in situ stress from K_0 towards K_a (noting that for excavations that are self-supporting, $K_a = 0$). Greater than expected movements usually result from insufficient stiffness of the retention system (wall, membrane and supporting struts or anchors) and/or insufficient rows and/or insufficient post-tension force applied to anchors or struts (to maintain near K_0 conditions). Such inadequacy can occur in the design of the retention system, poorly constructed anchors or by not staging the construction works appropriately.

Movement of the retention system walls away from the excavation (i.e. in the passive direction) results in an increase in the in situ stress above K_0. This usually only occurs close to the ground surface in soft clay and is usually due to the design specifying excessive post-tensioning force in the upper row of anchors or struts.

The amount of movement that occurs depends on the stress change that occurs and on the stiffness of the ground. It follows that K_0 and ground stiffness are also key parameters for any retention system design.

An appropriate ground investigation therefore must be able to not only identify the subsurface stratigraphy and ground water regime but must also allow the strength, stiffness and at rest pressures in the ground to be reasonably quantified.

3 RETENTION SYSTEMS IN SOIL

The analysis and design of retention systems in soil are relatively well understood with comprehensive guidance for designers being provided in CIRIA C760 "Guidance on embedded retaining wall design". Key inputs into the analysis and design include soil strength and stiffness, at rest in situ horizontal stresses, ground water conditions (and wall strength and stiffness).

Appropriate ground investigations in soil allow a suitable geotechnical model to be developed which identifies the thickness and general characteristics of each soil unit within the geotechnical model. Each soil unit is generally assumed to be homogeneous and isotropic with no difference between intact and mass properties. Strength and stiffness properties are usually assessed directly from small laboratory samples, indirectly from in situ testing or from experience. Soil structure if it is present is usually included in the assessment of the strength and stiffness parameters (e.g. a lower strength may be assumed if a clay is fissured).

K_0 is rarely measured. However, a prudent estimate of K_0 in soils can usually be made based on knowledge of the soil stress history and the strength and stiffness of the soil. By definition, K_0 must be between the two failure states (active) K_a and (passive) K_p, and generally in most near surface soil deposits K_0 is much closer to K_a than to K_p. It is rare for soils within the depth range of most retention systems to have locked in horizontal in situ stresses close to K_p as the usual soil formation processes do not allow stresses close to K_p to develop. Even if they do develop, the low stiffness of the soil allows elevated stresses to dissipate. Nevertheless, because the strength of the soil is relatively low, the in situ horizontal stress in most near surface soil strata is also generally relatively low. That is, the in situ horizontal stress in soil that a retention system needs to support is usually relatively low.

Another aspect of retention systems in soil is that the retention system elements (piles, walls, anchors, struts) are generally much stiffer (and stronger) than the soil. As a result, it is usually practical to design and build a retention system in soil to limit movements to tolerable levels.

4 CHARACTERISTICS OF ROCK MASS BEHAVIOUR

Many reference books and publications such as CIRIA C760 provide the key aspects of analysis, design and performance of retention systems in soil. Most, if not all of these key aspects, are also applicable to retention systems in rock and as such, these publications provide a fundamental framework for the analysis and design of retention systems in rock. However, there are some important and significant characteristics of rock masses that are not usually considered when designing retention systems in soil. These are summarised below.

4.1 ROCK FORMATION PROCESS AND IMPACT ON ROCK MASS CHARACTERISTICS

All rock masses comprise planes of weakness or discontinuities (e.g. bedding, joints, faults, fractures etc.), referred to as rock structure. The structure and in situ stresses present within a rock mass are dependent on their formation process and geological history. For example, basalt which forms from the cooling of lava is prone to shrink through the formation process creating persistent vertical joints or columns with typically low horizontal in situ stress, whereas sedimentary rocks, which form through layering and consolidation of sediment, are more likely to have persistent horizontal bedding planes with higher in situ stresses. The stresses in the rock mass can then change through tectonic activity subsequent to emplacement of the rock. This can result in compression or extension of the rock mass leading to the development of further structure, including joints, folding and faulting, which develops in orientations influenced by the direction of the stresses applied.

Regional tectonic compression of a rock mass typically results in a stress anisotropy, with high horizontal stress in one direction and a lower horizontal stress in the orthogonal direction. These stresses typically result in deformation within the rock mass ranging from folds and shears through to faulting and jointing depending on the ductility of the rock mass. In brittle rock masses, reverse or thrust faults, which are essentially a failure of the rock mass once its passive resistance is exceeded can form with strike perpendicular to the direction of the principal stress. Similarly, normal faults and shears can form at an orientation perpendicular to the minor in situ stresses. The high displacement which occurs along the fault planes or slip between joints and bedding can serve to reduce strength along discontinuities to residual levels (e.g. slickensided bedding planes).

The in situ stresses, strength and stiffness characteristics of discontinuities are further altered by weathering, erosion and anthropogenic changes (human impact) such as excavation (e.g. mining).

4.2 ROCK STRUCTURE

The presence of discontinuities within a rock mass not only reduces the mass strength and stiffness of the rock significantly below that of the intact rock, but also introduces significant strength and stiffness anisotropy into the rock mass, with the strength and stiffness of the rock mass varying in magnitude and orientation depending on the characteristics of the discontinuities, including their strength, spacing, persistence, orientation, roughness and infill and the scale being considered. For example, the strength and stiffness of a rock mass with widely spaced discontinuities will be significantly greater for a 2 m deep excavation (due to the very blocky nature of the rock mass at this scale) than for a 20 m deep excavation (where the rock block size is small compared to the overall depth of the excavation).

There is often large variation in the discontinuity properties within the same rock mass. For example, bedding planes which tend to be persistent over tens if not hundreds of metres in sedimentary rock masses can have strengths varying from the intact rock strength to a friction angle of less than 10° on smooth, slickensided surfaces. Similarly, the spacing, aperture and infilling of joints can vary significantly with location and orientation and as a consequence, the stiffness of the rock mass also varies significantly.

This makes assessment of rock properties for the analysis of retention systems a difficult and imperfect task. The orientation, location, persistence and type of discontinuities within a rock mass will have different impacts on a retention system depending on the orientation (strike) of the retention system relative to the major discontinuities. For example, in a rock mass where the strike of the bedding is north-south and persistent bedding dips to the west, then the bedding is unfavourable and favourable for the east and west faces, respectively, of a retained excavation. However, whilst the orientation of the bedding can be identified from an appropriate geotechnical investigation, the location of the weakest bedding planes within the rock mass, and which have the greatest impact on the design and performance of the retention system, cannot usually be reasonably known. As a result, a designer must consider a range of potential defect locations and strengths that are critical to the performance of the retention system.

For example, in Melbourne Australia, the siltstone which forms the bedrock comprises siltstone with interbedded fine grained sandstone. Bedding planes can be persistent over hundreds of metres and with strengths that vary locally on the same bedding plane from relatively high (essentially intact rock strength) to very low (about 12° on slickensided planes). A relatively large number of laboratory direct shear tests have been undertaken on bedding plane joints in the siltstone over a number of years. The results of these tests generally indicate friction angles in excess of 35° with a reasonable lower bound value of about 23° which is similar to the residual strength of the rock. On rare occasions, testing has indicated friction angles as low as 12° for slickensided bedding planes. Such slickensides are relatively common but are generally restricted in their persistence. That is, the areas of slickensiding are generally not extensive, and hence in most situations do not significantly impact the performance of a retention system. However, on at least three occasions that the authors are aware of, the presence of an extensive slickensided bedding with a persistence of over at least 50 m resulted in either a batter collapse or excessive movement of a retention system requiring emergency remedial action.

One example where this problem has arisen was for an anchored soldier pile and shotcrete retention system constructed in a siltstone rock mass with a locally elevated groundwater table (due to a leaking service) and containing persistent bedding striking approximately parallel to the east excavation face and dipping out of the face at about 25°. One of the bedding planes (out of many hundreds) near to the base of the excavation (see Figure 1) was slickensided over a significant length (in excess of 50 m).

Figure 1: Photograph showing a slickensided bedding plane which caused movement of the retention system of the east wall of the excavation in excess of design

Direct shear tests on large samples obtained by coring into the rock mass at different locations along the sliding bedding plane shown in Figure 1 (with the bedding aligned along the axis of the core) indicated shear strengths ranging from 12.5° to almost intact rock strength. One test conducted on a sample obtained from the north wall of the excavation indicated a relatively high initial peak strength and then a very rapid reduction to a friction angle of 18°. This very brittle behaviour is shown in Figure 2 (results of a direct shear test under constant normal stiffness conditions with initial normal stress of 370 kPa and normal stiffness of 50 kN/mm). The very brittle behaviour makes the assessment of the appropriate bedding strength to be adopted for design difficult.

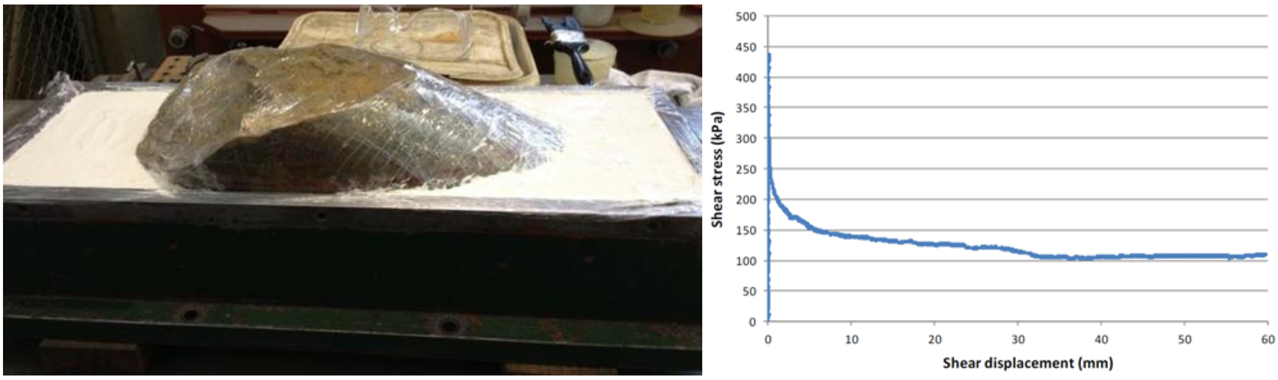

Figure 2: Photograph showing sample secured in bottom half of split shear box with plaster (left) and direct shear response showing very brittle behaviour (right)

In addition to the persistent weak bedding plane, an (unknown) sub-vertical weak structure was present within the rock mass at the northern end of the site which acted as a release plane allowing a large wedge of rock to rotate out of the face as shown in Figure 3. The locally elevated groundwater table and presence of a persistent slickensided bedding plane and sub-vertical weak structure were not known at the time of design and was considered in design.

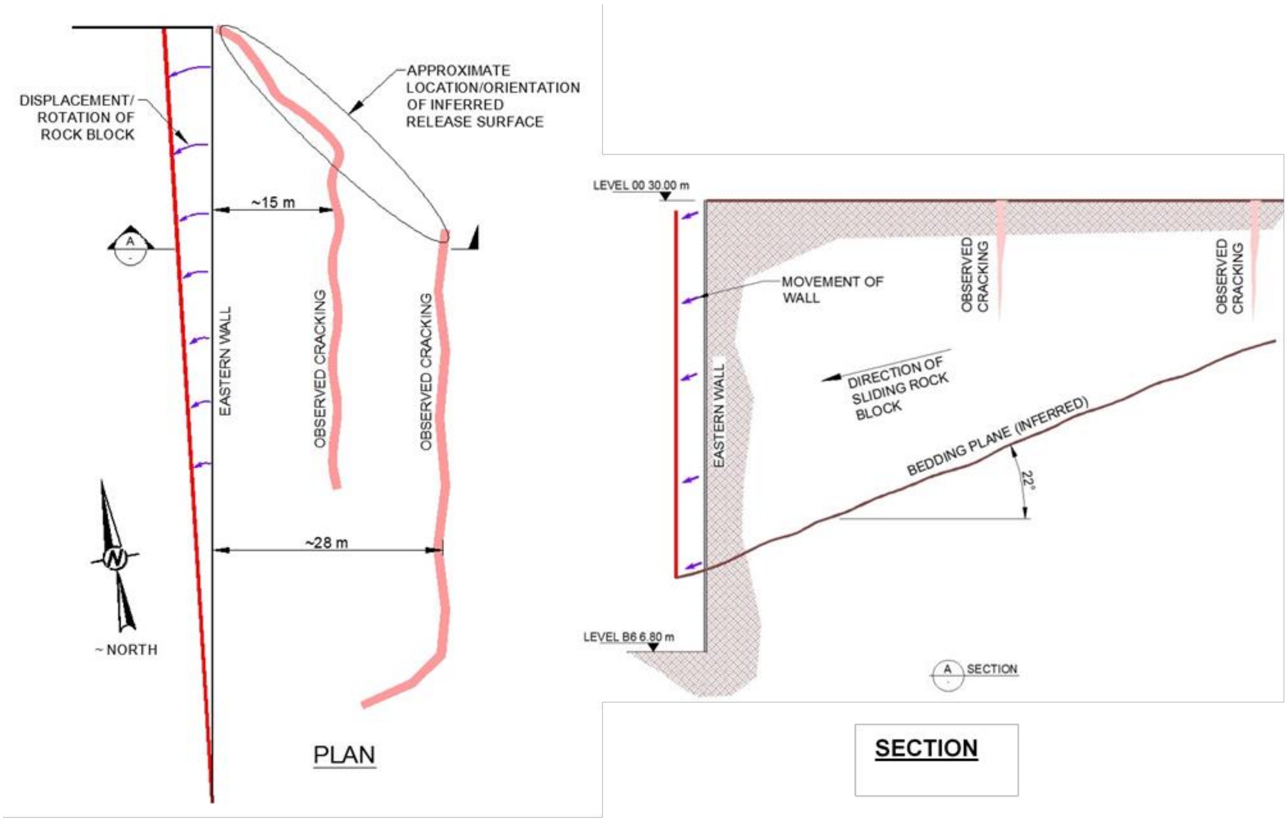

Figure 3: Mechanism of movement of east wall (plan, left and section, right)

The excavation at this site progressed without issue until the excavation was almost at full depth. The first sign of an issue was cracking and spalling of shotcrete between soldier piles at the very northern end of the east wall. Initially, this was difficult to explain as such damage would usually occur in the central portion of the wall. However, subsequent investigation indicated the spalling first occurred at the location of a subvertical release feature within the rock mass behind the north east corner of the excavation (see Figure 3). The first observation that a significant rock wedge had formed was provided by the soldier piles at the southern end of the east wall. Below a depth of about 20 m the rock was no longer in contact with the soldier piles, but above this depth the rock mass was in contact with the soldier piles. Closer inspection revealed the bedding plane on which sliding was occurring and this could be traced for at least 50 m to the north whereupon it was covered by shotcrete. As displacement monitoring was only being undertaken on a fortnightly basis there was no early warning of movement. Subsequent movement monitoring of the whole east face indicated that the wedge was moving as a block into the site. Emergency stabilisation measures (backfilling against the east face followed by installation of sub-horizontal bores to lower groundwater levels and installation of additional post tensioned rock anchors) were immediately employed. Movement of the rock wedge continued to occur until sufficient backfilling had occurred (where upon the east wall had displaced horizontally about 100 mm) and was not completely arrested until the additional rows of rock anchors were installed.

The above example focussed on bedding, which is often the most persistent and critical with respect to design of a retention system. However, other discontinuities of lower persistence (e.g. joints) can also have an impact. The orientation, spacing and persistence of such features relative to an excavation face defines the dimensions and geometry of rock wedges that are present and hence can be used to estimate the potential localised forces that need to be carried by the retention system (e.g. as may need to be carried by shotcrete between soldier piles, or for a larger wedge encompassing say a few piles only). Typically, the form of the rock wedges will change from one face of the retention system to another.

As a result of the rock structure the overall rock mass strength and stiffness is both anisotropic and scale dependent.

4.3 IN SITU HORIZONTAL STRESSES

The in situ horizontal stress present in rock masses is commonly developed through tectonic activity and remains locked in until excavation, weathering or erosion causes a change in the magnitude or orientation in the in situ stress field. In situ horizontal stresses can vary between active and passive pressures depending on the stress history of the rock mass. Faults, shear zones, erosion and weathering can all modify and lead to inhomogeneity of the stress field such that whilst there may be a consistent stress magnitude and orientation at the regional scale, at a local or project scale the in situ stress may vary, in some cases significantly from the regional stresses.

The highest in situ horizontal stresses generally occur in compressional tectonic settings and can be close to the passive resistance of the rock mass. The highest stresses will typically be present in the strongest and stiffest horizons within the rock mass. In a sedimentary rock mass for example comprised of siltstone and sandstone, the more competent (and higher stiffness) sandstone may attract higher stresses. More weathered ground might be encountered at shallow depth and may attract lower stresses, or there may be modification to the stress field around existing excavations. All of these factors can serve to produce local stress conditions different from regional conditions.

Undertaking excavations in areas with high in situ horizontal stresses can be problematic – not because of instability of the walls of the excavation which may be self-supporting, but because of the relatively large (and “stable”) movements that can occur due to the relaxation in in situ horizontal stress caused by undertaking the excavation.

For example, the authors recently had experience with the excessive movement of one wall of a deep basement excavation in medium to high strength metamorphosed rock due to unexpectantly high in situ horizontal stresses. The retention system comprised anchored soldier piles supporting the upper weaker soil and rock units and pattern rock bolts below. The rock mass was blocky in nature with relatively high Geotechnical Strength Index (GSI, Hoek *et al*., 2013). The foliation with the rock mass, whilst reasonably persistent, was favourable for the excavation face that suffered the unexpected movement. Like the previous example above the works were progressing in accordance with the design performance until the excavation encountered high strength rock below about 12 m depth. Automatic hourly displacement monitoring of the walls of the excavation indicated very low horizontal and vertical movements up until the excavation extended below the toe of the soldier piles into medium strength rock where upon increased movements of the retention system, but still in accordance with design, occurred. As the excavation extended into higher strength rock the lateral and vertical movements of the retention system significantly increased. At the completion of the excavation to full depth the wall had moved approximately 125 mm horizontally with settlement behind the wall of up to 90 mm.

After the onset of the increased movements, it was thought that the movement may be due, at least in part, to the formation of a large unstable rock wedge. As a result, back filling was undertaken against the face, and on the recommendations of a review consultant, additional rows of rock anchors were installed over a portion of the face already excavated and over the full length of the face as excavation proceeded. Analysis undertaken to design the additional support indicated that if an unstable rock wedge was the mechanism causing the movements, then additional movements following the installation of the rock anchors should be minimal and a factor of safety of 2 was achieved on wedge sliding.

However, although we considered these remedial measures to be prudent, it was not clear that an unstable rock wedge had formed or was the cause of the movements. The movements that had occurred and our observations of the site were inconsistent with this mechanism, The movements occurred relatively uniformly over the full length (in excess of 100 m) and excavated depth of the wall and effectively ceased soon after that excavation flitch was complete. There was no evidence of cracking or distress in the shotcrete or exposed rock faces and no significant damage was observed in existing buildings and assets located immediately adjacent to the top of excavation (although these had been subjected to significant movements). Lift off testing of anchors indicated anchor loads that were not consistent with a wedge mechanism. In addition, subsequent survey monitoring of the area around the site indicated a relatively widespread movement consistent with those due to stress relief.

Through further investigation and due to previously unavailable information becoming available, it became apparent that very high in situ horizontal stresses were present within the rock mass in this region. Further evidence of high in situ stresses being the cause of the movement is obtained from the movement that continued to occur following the installation of the remedial anchors. In fact, movements continued at an increased rate with depth with each excavation flitch, which was not consistent with the installation of the additional rows of anchors and the wedge hypothesis. The additional anchors appeared to have little if any impact in reducing the movements.

In ground conditions with relatively high in situ horizontal stresses, the anchor/strut forces required to limit lateral movement of the retention system can be impractical to achieve. For example, to balance an in situ horizontal stress of 1 MPa would require anchors on a 2.5 m x 2.5 m grid spacing each post tensioned to 6.25 MN. Under such circumstances, an option may be to accept that large "stable" horizontal movements are likely to occur and to design the retention system only to prevent instability rather than movement.

In ground conditions where the at rest pressure is close to K_p, any excavation will reduce the vertical effective stress acting on the base of the excavation and this may be sufficient to initiate passive failure of the floor of the excavation. Whilst such failure may not be directly observed during the works because of the ongoing excavation works (which removes failed material), it can result in high lateral movement of the otherwise stable walls of the excavation.

4.4 ROCK MASS PERMEABILITY

Most intact rock is relatively impermeable compared to the rock mass which is dissected with open fractures and discontinuities. In rock masses the groundwater flows are primarily restricted to the open fractures and discontinuities. As a result, only a relatively small volume of water is required to completely fill fractures. Groundwater levels can therefore rise quickly in rock with consequence for retention system design. In such instances, installation of sub-horizontal drains drilled into the excavated face to intersect primary water bearing fractures and features can significantly reduce the potential for adverse groundwater level rise impacting on the retention system.

5 CRITICAL GROUND PROPERTIES FOR DESIGN

As set out above there are a number of key geotechnical inputs that need to be quantified to enable a reasonable and prudent analysis and design of a retention system to be undertaken. These include the subsurface stratigraphy both within and outside the site, the characteristics (orientation, spacing, persistence) of the discontinuities within the rock mass, the strength of the discontinuities (especially bedding if present), the rock mass strength and stiffness, the ground water regime and the in situ horizontal stresses present within the rock mass.

A necessary starting point is to understand the geological and geomorphological history of site, the anthropogenic activity and its regional geological setting. An understanding of this information provides important insights into the likely subsurface conditions including the potential variability, the presence of significant geological features such as paleo channels, dykes, faults and folds and other rock mass properties including jointing patterns, weathering profile, the magnitude and direction of in situ horizontal stresses and groundwater conditions (including the potential for contamination plumes). A conceptual ground model would be developed at this point.

The scope of the geotechnical investigation can then be developed based on past experience in the vicinity of the site and the insights obtained from developing a conceptual ground model as set out above. Typically, an intrusive geotechnical investigation would be undertaken with boreholes drilled within the site and/or just outside the site boundaries. The number and depth of boreholes should be sufficient to reasonably and confidently define the subsurface geology (including different geological units and horizons of weaker, stronger and more weathered or fractured materials within each unit) and the groundwater regime. If there are known major geological features (e.g. buried paleo channels, major faults, dykes or shear zones) then these need to be investigated in sufficient detail to define their location, extent, geometry and material characteristics.

High quality drilling (e.g. triple tube core drilling) is required so that accurate information on rock mass conditions can be assessed without having to allow for artificial breaks in the core caused by the drilling. Even though auger drilling is often practical in very weak rock, it is preferable to commence coring as soon as is practical to obtain reasonable samples in the weak upper rock materials. Comprehensive borehole logs recorded by experienced and capable personnel and containing detailed information on discontinuities and other characteristics of the rock is essential. Auger drilling in rock provides little if any useful information and should be discouraged.

In rock masses for which the rock formation process indicates that persistent and pervasive discontinuities may be present within the rock mass (e.g. a bedded sedimentary rock), it is necessary to identify key features that could assist in estimating the shear strength of these discontinuities including persistence, orientation, spacing, infill and aperture, noting that these will vary through the rock mass and sufficient data need to be obtained to assess the variability and to estimate reasonable and prudent characteristic values. Orientation and spacing information can be obtained from nearby rock exposures and by undertaking borehole imaging, but noting that as boreholes are generally vertical, discontinuity information from borehole imagery is unlikely to provide a reasonable representation of sub vertical discontinuities or to provide an indication of the persistence of the discontinuities. In some instances, it may be prudent to utilise inclined boreholes, especially when delineating major geological structures.

Although the strength of discontinuities is critical to the analysis and design of a retention system, it is rare, and often of little value to undertake strength testing of discontinuities. This is due to the usually wide variation in discontinuity strength at the scale samples are tested. It is usually sufficient for such information to be assessed from visual observation of the discontinuities in the core and past experience. Careful observation of the core is therefore required. Some slickensided bedding planes can be very thin (< 1 mm) and difficult to observe in the core (see Figure 4), and yet it only takes one such plane to be present within the excavation depth, be persistent, with an unfavourable orientation to cause significant stability issues for a retention system. However, it is not reasonably possible to assess the extent and persistence of such bedding planes from rock core, leaving the designer with a difficult decision as to what to include in the analysis of the retention system. This is where combining borehole indications with a thorough understanding of the geological setting is vital.

Figure 4: Photograph of section of core which contained a slickensided bedding plane which caused movement of a retention system shown in Figure 1

Recovered rock core should be tested to measure the uniaxial compressive strength (UCS) of the rock. This together with information on the rock structure and application of a suitable rock mass classification system (e.g. GSI, Hoek *et al.*, 2013) is usually sufficient to provide a reasonable estimate of rock mass strength and a likely range of rock mass stiffness values for design, especially for higher strength rocks. Such approaches must be used with caution in weaker rock masses with GSI less than about 30. For these weaker and more fractured rock masses, additional down hole testing using a high pressure pressuremeter is recommended to provide a better estimate of rock mass stiffness. There is usually little value in undertaking pressuremeter tests in higher strength (e.g. UCS greater than 10 MPa unless the rock mass is highly fractured) massive rock masses as the stiffness of the rock measured in the test is likely to be greater than can be reasonably measured by the instrument and is usually of sufficient magnitude that it has little if any impact on the results of an analysis of the retention system.

In geological environments where in situ stresses are likely to be elevated, it may be prudent to undertake in situ stress measurements using borehole instruments that may utilise for example hydofracturing or stress relief methods. It is important however to correlate the in situ stress measurements with the regional geology and the mass stiffness of the rock at the location the measurements have been taken. This is because the processes through which the in situ stresses develop in the rock mass result in the stiffer horizons having a higher horizontal stress than the horizons with lower mass stiffness (even though they may be adjacent to one another). In situ stress distribution is also affected by topography and major structures such as faults.

Even in seemingly benign in situ stress environments, horizontal in situ stresses can be significantly higher than expected or indeed used traditionally in design and analysis. For example, for many years it was thought that the horizontal in situ stresses in the siltstone forming the bed rock below Melbourne were relatively low. However, recent stress measurements for a major tunnelling project indicate an anisotropic in situ horizontal stress field with at rest earth pressure K_0 of more than double that generally assumed in the past (e.g. in highly weathered siltstone typical measured values of K_0 are about 0.6 – 2.0 in both the east west and north south directions whereas corresponding values in slightly weathered rock are 1.5 – >4.0 and 0.75 – >2.0 respectively). The anisotropic in situ stress field is consistent with the regional tectonic setting in which the southern part of the Australasian Plate is subject to east-south east to north-north west tectonic compression.

Another example is in the much stronger rocks of the Neranleigh-Fernvale Beds underlying parts of the CBD of Brisbane. The Neranleigh-Fernvale formation comprises predominantly phyllite with intact strengths of slightly weathered, near surface material in excess of 30 MPa. The tectonic stress history in the area is one of north east to south west compression which has resulted in shallow angle reverse faults striking approximately north west to south east. Recent stress measurements at relatively shallow depth within the depth of deep basements indicate very high horizontal in situ stresses (several MPa) in the north east south west direction with measured K_0 values of 6 to 8 with the high stresses appearing to be related to major fault structures.

6 ANALYSIS

6.1 MODELLING ROCK STRUCTURE

It is clear from the preceding discussion that the rock mass structure needs to be appropriately accounted for in any analysis of retention systems in rock. There are many non-linear numerical modelling analysis software packages available that are capable of modelling the rock mass structure and other aspects of rock masses. These software packages adopt either a continuum or discontinuum approach. The continuum approach (e.g. PLAXIS 2D and 3D) tend to be the most widely used for retention system design. Discontinuum approaches (e.g. UDEC and 3DEC), although better suited to rock mass modelling are not routinely used for retention system design. This may be because it is usually difficult at design stage to adequately quantify the rock mass structure, and modelling of structural elements (e.g. piles and anchors) in such tools may be more limited. In any case, experience has shown that when a continuum approach is used appropriately, it provides a reasonable estimate of retention system performance.

When developing analysis models using a continuum approach, the methodology adopted to simulate the rock structure varies from engineer to engineer, but most methodologies can be categorized into two main types. Either the rock structure is included in an assessment of the mass strength and deformation properties of the siltstone (continuum model) resulting in lower strength and deformation properties, or alternatively less conservative parameters are adopted for the rock mass and a limited number of distinct discontinuity planes are included directly in the analysis model at critical locations (partial discontinuum model).

The available continuum based software packages offer a range of constitutive models suitable for use in both continuum and partial discontinuum approaches. These models range from simple Mohr Coulomb models to hardening models in which different loading and unloading performance can be modelled. All of these models generally assume the rock mass is isotropic with respect to strength and stiffness.

A continuum based constitutive model that allows anisotropic strength and stiffness to be modelled is provided by ubiquitous joint models (referred to as the Jointed Rock model in PLAXIS). This model effectively models the rock mass as containing an infinite number of weak planes in up to three different specified orientations. The strength and stiffness of the rock mass in the specified directions is assumed to be equivalent to that of the discontinuities in those directions. The intact rock is considered to behave elastically.

An appropriate choice of constitutive model and rock properties (including in situ stress) used in the analysis of the retention system is key to obtaining reasonable estimates of performance of the system. For this reason, it is the authors' opinion and experience that such analyses should be undertaken using a constitutive model that captures the important aspects of the rock behaviour along with "prudently conservative best-estimate" properties.

For a continuum model to provide a reasonable estimate of performance, the rock mass must behave as a coherent mass (or continuum) and its performance must not be dominated by planes of weakness. Whether this is realistic or not depends on the scale of the problem relative to the rock structure and the orientation of the rock structure. Hoek and Bray (2004) illustrate this in the example shown in Figure 5.

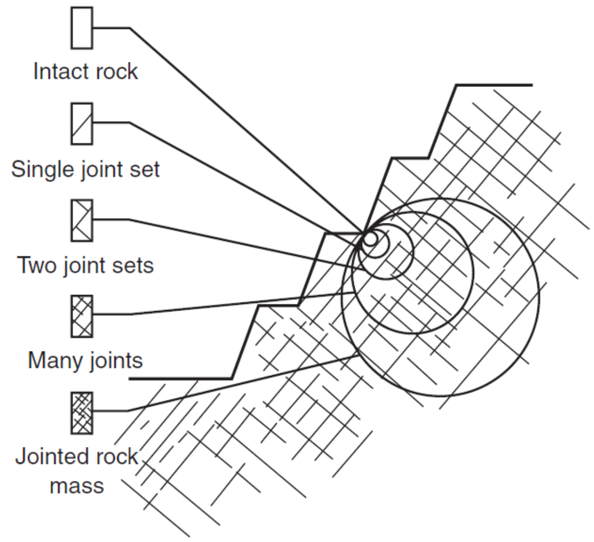

Figure 4.1 Idealized diagram showing transition from intact rock to jointed rock mass with increasing sample size.

Figure 5: Figure 4.1 of Hoek and Bray (2004) showing scale and rock mass behaviour

If the rock mass performance is dominated by one (or more) set of discontinuities then this set of discontinuities needs to be explicitly modelled in the analysis (partial discontinuum approach). Usually for an excavation face it is one set of discontinuities which is critical to the performance of the retention system (e.g. bedding), although other discontinuity sets might provide release planes or impact on the form of the failure wedge.

Haberfield (2017) provides an example of an analysis of a 25 m deep excavation supported by a retention system in a rock mass that contains weak persistent bedding striking north south and dipping at 25° to the west. A summary of this example is included below.

On the east face of the excavation, the bedding is unfavourable and will dominate the behaviour of the rock mass and hence the bedding needs to be discretely modelled. However, on the north, south and west faces of the excavation, the bedding is either favourable or neutral and does not impact significantly on the performance of the retention system and hence need not be explicitly modelled in the analysis.

The analysis was undertaken using PLAXIS 2D for three different constitutive models, Hoek Brown, Mohr Coulomb and the Jointed Rock model.

The rock mass parameters for the continuum approach adopted for the north, south and west faces were calculated based on an estimate of the Hoek and Brown GSI properties (Hoek *et al.*, 2013) of the siltstone for the scale of the rock mass under consideration, i.e. the full 25 m depth of the excavation. Equivalent Mohr-Coulomb parameters for effective rock mass cohesion and friction angle were calculated at a representative vertical stress. For the Hoek-Brown model, the GSI parameters were used directly. The same Mohr-Coulomb properties were used for the jointed rock model as appropriate, along with bedding plane friction angles of 35° for the Serviceability Limit State (SLS) case and 23° for the Ultimate Limit State (ULS) case (see below).

For the partial discontinuum approach, potential low strength bedding planes were included in the analysis using interface elements. The positions of the bedding planes were selected to represent the worst possible location with respect to the performance of the retention system i.e. at base of each excavation flitch prior to anchor installation. As for the jointed rock model, the SLS case assumes a bedding plane friction angle of 35°, whilst the ULS case considered a minimum of three low strength bedding planes with a friction angle of 23° are present. Figure 6 shows the PLAXIS model used for the critical east face (partial discontinuum model) of the excavation.

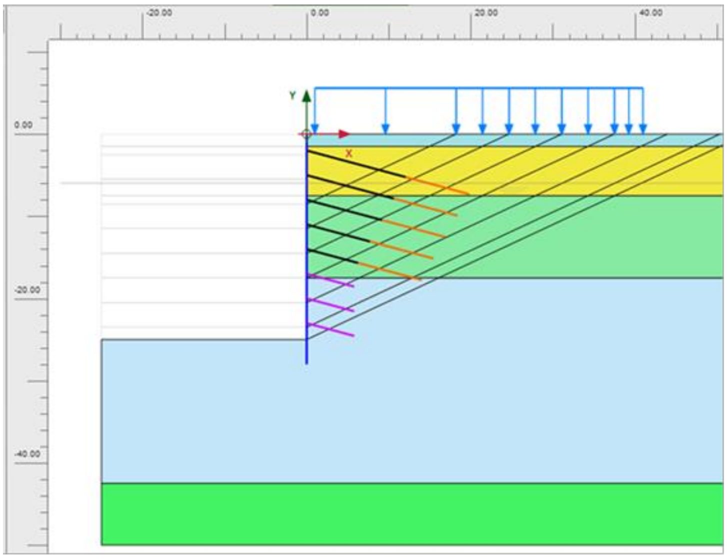

Figure 6: Partial discontinuum PLAXIS 2D model adopted for the critical east face

The Mohr-Coulomb rock mass parameters used in the discontinuum model were calculated based on an estimate of the Hoek and Brown GSI properties (Hoek *et al.*, 2013) of the siltstone for the smaller scale of the rock mass between the low strength bedding planes (i.e. about 5 m). This resulted in higher rock mass strength and deformation parameters than assessed for the Mohr-Coulomb continuum model. All other inputs into the analyses were identical.

As indicated in **Error! Reference source not found.**, the analyses using the different approaches and constitutive models showed significant differences in the results of the analysis in respect to the calculated lateral displacements, pile bending moments and shear forces and total anchor and bolt load. The jointed rock model failed to converge for both SLS and ULS analyses. It was concluded that a significantly more robust retention system was required to obtain convergence using the jointed rock model.

Table 1: Comparison of calculated results

Structural Reactions	Continuum		Discontinuum	
	Mohr-Coulomb	Hoek-Brown	SLS	ULS
Max. lateral disp. (mm)	29	48	13	20
Pile max. bending moment (kNm/m)	298	346	240	333
Pile max. shear force (kN/m)	260	297	660	860
Total anchor/bolt force (kN/m)	800	950	750	850

The different structural actions obtained for the east and other faces (i.e. between the partial discontinuum and continuum approaches) arise because of differences in the calculated deformation patterns. Figure 7 compares the relative deformation patterns for the partial discontinuum and continuum approaches assuming a Mohr Coulomb constitutive model.

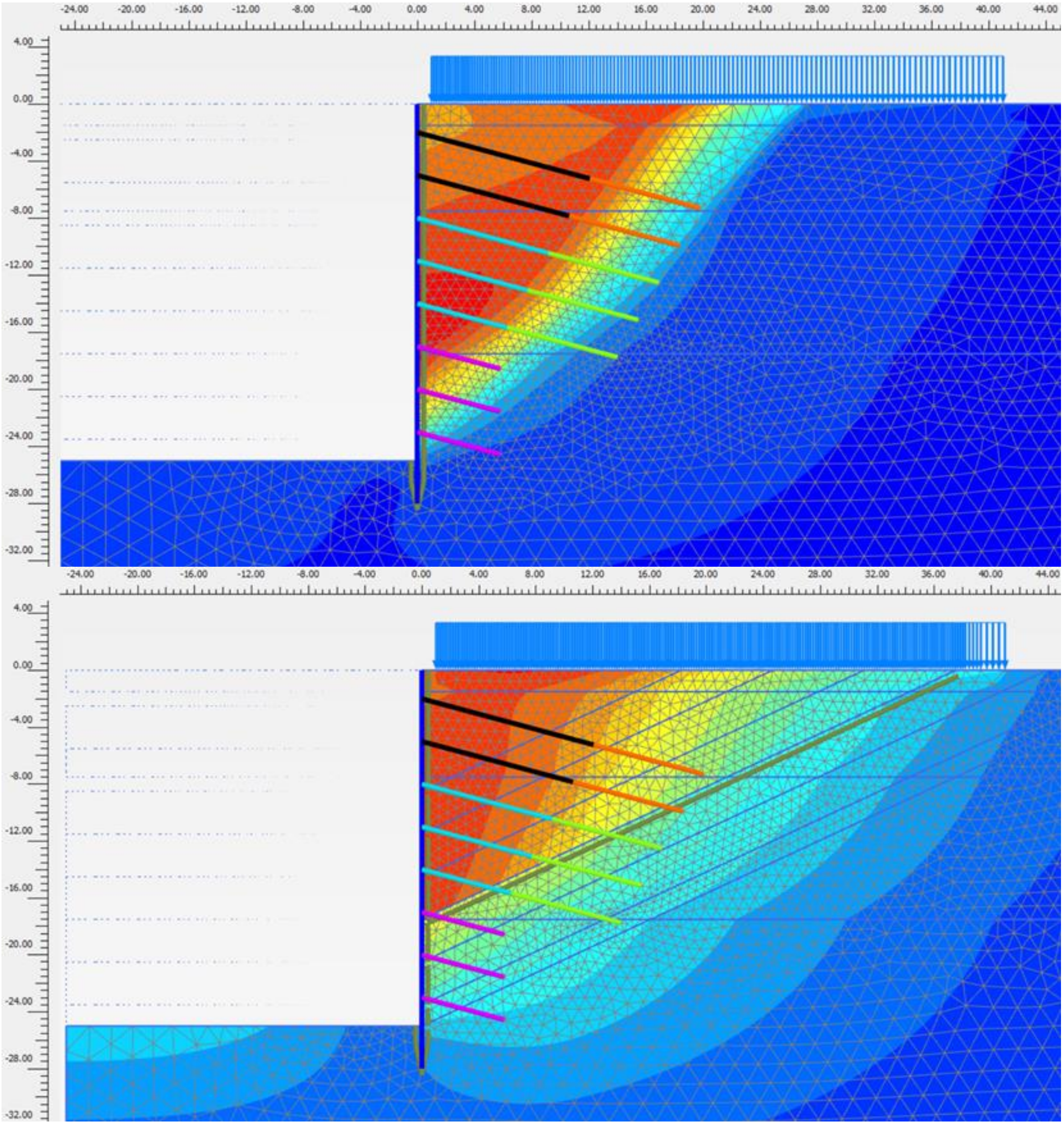

Figure 7: Comparison of deformation patterns using partial discontinuum (top) and continuum (bottom)

The different structural actions obtained using the different constitutive models arise because of differences in the strength envelopes adopted. In the authors' experience the Jointed Rock model generally provides a significant underestimate of rock mass performance in this application as evidenced by proven satisfactory performance of many retention systems designed using a Mohr Coulomb constitutive model. It is also the authors' experience that in general the Mohr Coulomb constitutive model indicates better retention system performance than provided by the Hoek Brown constitutive model. This is likely a result of the relatively high curvature of the Hoek Brown model near the zero stress origin (i.e. due to its near parabolic shape) compared to the linear Mohr Coulomb constitutive model. Whilst a near parabolic shape to the failure envelope appears to be reasonably representative of the strength behaviour of stronger rocks, weaker rocks (i.e. those with low GSI) tend to display a near linear strength envelope at lower stress levels that is closer to a Mohr Coulomb criterion (Johnston, 1985). This indicates that a linear strength envelope as provided by the Mohr Coulomb constitutive model may be better suited to weaker rocks and the Hoek Brown model to stronger rocks.

The range in results obtained from the different approaches and models used in this example has significant consequences for the design of the retention system. The continuum model results indicate significantly higher anchor forces, bending moments and displacements than the partial discontinuum model. It is therefore likely that a retention system design based on a continuum approach, in this case, would likely require larger diameter piles or piles with a greater percentage of reinforcement and higher capacity anchors. It is also likely that anchor lock-off loads may need to be increased to reduce lateral displacements. The continuum model also indicates significantly lower shear forces in the piles than calculated using a partial discontinuum model. This may indicate that piles designed using a continuum approach have insufficient shear capacity.

It is not known which of the above models provides the most accurate assessment of the performance of the modelled retention system. However, the authors have used the partial discontinuum approach to design many retention systems in bedded siltstone and all but one has performed satisfactorily with movements generally within design expectations. The one exception was for an anchored soldier pile and shotcrete retention system constructed in a siltstone rock mass (as described above).

The example set out immediately above does not consider a high groundwater table nor the presence of a persistent 12° bedding plane (the 2D plane strain nature of the analysis effectively assumes the presence of release surfaces at either end of the excavated face). Inclusion of both of these into the analysis will significantly affect the calculated results. The most likely outcome is that the retention system adopted above will not be satisfactory and anchor and pile support will need to be strengthened. Both the continuum and partial discontinuum approaches described above can satisfactorily model high groundwater conditions. However, the presence of a slickensided bedding plane cannot be reasonably modelled in the continuum models.

A question arises as to whether or not a retention system should be designed to accommodate the rare occurrence of an extensive, slickensided bedding plane. Such bedding planes are almost impossible to identify through ground investigation, but their inclusion in the analysis significantly increases the cost of the retention system, possibly making the proposed development uneconomic. It may indeed be impractical or cost prohibitive to design for low angled bedding with very low friction angles as the efficiency of post tensioned anchors in particular (but also applies to struts) in such conditions is significantly reduced. The approach that has been adopted by the authors is to not include such very low strength bedding planes in analysis, but to rely on monitoring and an observational approach, with the aim of identifying a potential problem early so that strengthening measures can be implemented if required.

6.2 MODELLNG IN SITU HORIZONTAL STRESSES

A common assumption in retention system design is that there is a uniform but increasing horizontal in situ stress field with depth, with the magnitude of the in situ horizontal stress related to the vertical stress by K_0. It is also common for engineers to adopt the maximum horizontal in situ horizontal stress and lower bound estimates of rock stiffness.

However, high in situ horizontal stresses are usually developed as a result of tectonic movements, wherein the rock mass is compressed in one direction. The impact on the rock mass is one of uniform strain with depth not uniform stress, and as a result, higher horizontal stresses will develop in the stiffer beds within the rock. It is not appropriate to adopt upper bound in situ stresses and lower bound rock stiffness values in an analysis of a retention system. Upper bound stresses occur within the stiffest rock, and appropriately high stiffness values should be adopted.

A more realistic method of modelling the initial in situ stress field is therefore not to use K_0, but to compress the rock mass in the horizontal direction by applying a constant displacement at one or both vertical boundaries of the analysis model to impose a uniform strain field on the rock mass. The magnitude of the boundary displacement applied depends on the width of the analysis model, but it needs to be set at a value that causes the known (or estimated) in situ horizontal stress to develop at the appropriate depth. Lochaden and Haberfield (2019) describe such an analysis that they carried out for a deep basement in horizontally bedded Sydney sandstone.

In situ horizontal stress fields within rock are usually anisotropic and this needs to be accounted for when modelling different faces of the excavation.

In situations where the horizontal stress field is very high, close to K_p, software packages based on achieving equilibrium (e.g. finite element methods such as PLAXIS) may not converge. This is because any reduction in vertical stress due to excavation results in passive failure at the base of the excavation. For such situations programs such as FLAC which do not rely on reaching internal equilibrium may be better suited.

7 DESIGN PHILOSOPHY

The above example highlights the importance of adopting realistic soil models and properties in analysis of retention systems. Results of such analyses can be seriously erroneous if inputs into the analyses are not reasonable and appropriate.

There are a number of standards, codes and local statutory rules which require various factors to be applied to soil and rock properties for the design of earth retaining structures. Many relatively common design approaches require application of reduction factors to elements that provide resistance (e.g. rock mass strength) and load factors to disturbing elements (e.g. rock weight). However for retention systems and retaining walls, rock weight also contributes to rock mass strength, and hence applying a load factor on bulk unit weight of the rock will add to the strength and resistance of the rock. This is clearly illogical if used in an analysis of a retention system.

The authors therefore question the practice of applying factors to reduce and increase strength parameters and loads (e.g. bulk density), respectively, in the analysis of retention systems for ULS considerations.

As stated above, it is the authors' opinion that analyses of retention systems should be undertaken using "prudently conservative best-estimate" properties. This applies to both SLS and ULS analyses, both of which should be undertaken. The SLS analysis should be undertaken assuming reasonably likely scenarios/conditions and prudently conservative properties. ULS analysis should be undertaken for the worst credible conditions but still using prudently conservative properties.

For example, an SLS analysis of a retention system should consider prudent, reasonably likely ground conditions, groundwater levels, loading and construction sequencing. The ULS analysis should consider credible worst-case conditions, e.g. accidental overdig, temporary high groundwater table (e.g. due to water main failure), unfavourable, persistent slickensided fissures or bedding, earthquake loading etc.

The structural reactions obtained from these analyses must then be factored up for design. Different factors should apply to the SLS and ULS reactions, and the maximum factored reactions adopted. Nevertheless, these analyses are usually still assumed to represent SLS conditions and therefore an appropriate load factor (e.g. 1.35) needs to be applied to the calculated structural reactions for design.

The guidance provided in CIRIA C760 (CIRIA, 2017) in relation to design philosophy is largely consistent with that set out in Eurocode 7 (British Standards Institution, 2004), i.e. adopting both Design Approach 1 (applying partial factors to increase permanent and unfavourable variable loads) and Design Approach 2 (applying partial factors to decrease soil strength and increase permanent and unfavourable variable loads). It is the authors' understanding that this approach may be revised in subsequent revisions to Eurocode 7.

Another design decision that needs to be made is the allowable movement of the retention system. A relatively light retention system may prevent failure but movements will be greater than if a stiffer retention system (comprising larger diameter piles and or higher anchor loads) is adopted. For many basement excavations in good quality rock, in situ stresses can be relatively high and batters (at least on favourable faces) can stand vertically without support, or at least with only limited support provided by rock bolts to mitigate localised failures. However, due to the high in situ horizontal stresses very high anchor loads are required to limit movements to normally acceptable levels. Under such circumstances, consideration should be given to providing minimal support provided larger than usual displacements can be tolerated. Such larger than normally accepted movements should be properly communicated so that they are not mis-interpreted as impending failure.

8 ROCK ANCHORS

Whilst the retention system wall (piles, diaphragm wall etc) are the main structural component of a retention system, the majority of the support is provided by the anchors and/or props. The piles primarily act as a "plate" to spread the anchor and prop loads over the excavated face. For this reason, the performance of the retention system is largely dependent on the performance of the props and anchors.

The authors' experience in Australia is that post tensioned anchors are preferred over the use of props and that very few retention systems in Australia are supported by props. However, the use of props is increasing due to issues associated with obtaining permission to install anchors beyond property boundaries.

The performance of post tensioned anchors is highly dependent on how they are installed and tested. In Australia, ground anchors are usually installed under a design and construct contract with contractors required to achieve stated loads via proof loading tests. However, the authors experience is that the quality assurance and quality control associated with ground anchor installation is often inadequate and requires improvement. Some anchor contractors believe it is not necessary to proof load every anchor, nor to keep detailed records of installation and proof loading, and as they are rarely asked to produce this information they see no reason to record it. There is also a general misunderstanding with respect to bond length and free length, with a common practice being to “grease and sheath” the free length but grout the entire length of the anchor. If this is done then the proof loading test is of no value as the resistance over the full grouted length of the anchor is measured rather than just the resistance over the bond length.

The authors generally recommend the following in respect to ground anchors:

- due to the axial flexibility of anchors and the potential for “unzipping”, the bond length of anchors should be no less than 3 m and no greater than 10 m to 12 m;
- for anchors installed along excavation faces unaffected by adverse, persistent discontinuities (e.g. bedding) a free length extending 1 m beyond a plane extending up from the base of the excavation at an angle between 45° and 45° + φ/2 (to be assessed by the designer on a case-by-case basis, where φ is the internal angle of friction), or above better quality rock that is assessed to be self-supporting, is appropriate;
- for anchors installed along excavation faces that are impacted by adverse, persistent discontinuities, the free length needs to extend beyond a potential bedding plane that could impact the wall. For example, the free length of the upper most row of anchors needs to extend beyond a potential bedding plane located at 0.5 m below the second row of anchors and so on for subsequent rows. At every stage of the excavation, there must be sufficient bond length beyond the critical bedding planes to support the potentially unstable wedge;
- excavation must not proceed more than 0.5 m below the level of any post tensioned anchors prior to their installation and tensioning;
- all post tensioned anchors are secondary grouted and the minimum free length is maintained to allow post tensioning of the anchors. This may require some grout from the initial grouting stage to be washed out prior to tensioning and the secondary grouting stage. For the reasons set out above, the authors consider sheathed anchors grouted over the full length are not satisfactory for post tensioning;
- the anchor contractor is to provide a detailed design and construction methodology statement for review by the designer prior to the commencement of anchor installation;
- all anchors are proof loaded to confirm that the stated loads are achieved. As a minimum all anchors to be proof loaded to 125% of the design working load and held for 15 minutes with records of the applied load and anchor elongation made at specific stages of the proof loading; and
- full quality assurance and quality control documentation, including proof loading results, length of free zone following wash out, length of grouted zone following wash out, anchor declination, anchor lock off load, etc to be provided for every anchor and be available to the designer if requested.

9 MONITORING

The appropriate undertaking of the ground investigation and analysis of the retention system does not negate the requirement for an appropriate movement monitoring system. The extent of such a monitoring system is dependent on a number of issues, including the magnitude of the retained height and the risk to adjacent structures and services. As a minimum, a monitoring system would likely include target markers at capping beam level which are manually surveyed. More sophisticated monitoring systems may include inclinometers, a series of target markers placed along the retained height as excavation continues, and monitoring of adjacent structures and services, with surveying of target markers undertaken at regular intervals using automated total stations. It is critical that target markers at capping beam level are installed prior to excavation works.

The monitoring system must be accompanied by guidance on appropriate trigger levels and the associated action to be taken should these movement limits be reached. Guidance should also be provided on the frequency of movement monitoring. Although often ignored by engineers, it may be useful to relate the trigger levels to the degree of excavation which has taken place at that time, such that early warning of movement which is not consistent with the design is possible. The results of numerical analyses which consider the staged construction sequence may be useful in this regard. However, the accuracy and precision of the monitoring instruments must be carefully considered. Furthermore, low trigger levels which are unwarranted are likely be exceeded, and to cause both program delays and frustration to those involved in the construction of the retention system.

10 CONCLUDING REMARKS

This paper summarises the authors' experience in the design of retention systems in rock, which differs significantly from those in soil. A number of the critical issues associated with the design of retention systems in rock have been discussed, including those related to the appropriate characterisation of the ground, design tools and design philosophy. Reference has been made to a number of case studies, including those with which the authors were involved and which did not perform as anticipated. Appropriate monitoring systems were critical in identifying the divergence of the true behaviour of the system from the anticipated behaviour.

11 ACKNOWLEDGEMENTS

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