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Ground support methods for vertical development

CG Scott *pitt&sherry, Australia*

D Sidea *pitt&sherry, Australia*

SJ Masters *pitt&sherry, Australia*

Abstract

Vertical development is an integral part of underground mining. At a minimum, underground operations require vertical development for ventilation and secondary means of egress; many deep mines also require haulage shafts for material extraction. Vertical development can be excavated by various methods with raiseboring becoming one of the most common methods in Australian underground operations due to its speed, efficiency and safety. Other excavation methods for vertical development include blind boring, blind sinking, strip-and-line and vertical shaft sinking machine (VSM). Access for the installation of ground support varies depending on the excavation method.

Vertical development is often required to be functional over many years (20+). Once commissioned, it is often difficult or impossible to re-access shafts for rehabilitation support. The decision to support and the type of support, if required, can be critical to the shaft's operational success. The decision to support may be based on the performance requirement of the shaft, availability of access and anticipated stability issues. Stability issues ranging from near-surface weathered material, isolated poorer zones in more favourable rock masses and competent rock/high stress to base of shaft geometry are considered in this paper. An individual shaft may have several areas that require support and may require different assessment approaches.

In this paper, three main zones of vertical development are considered for stability and support design; the collar (near-surface poorer quality weathered zone/soils if excavated from surface), broken zones/poor ground conditions and the base of the shaft development. Many methods of assessing shaft behaviour and support requirements exist. This paper discusses typical shaft ground support types and selected methods of stability and support assessment, types of support, loading conditions and examples for various excavation methods/ground conditions.

Keywords: *ground support, vertical development, shafts*

1 Introduction

The ground support systems available for use in vertical development are highly dependent on the method of excavation chosen. Variables that need to be considered in the selection of an excavation method include top-down/bottom-up, entry/non-entry and remote support application/personnel access for handheld installation of support. A selection of common excavation methods and corresponding available general support options are presented in Table 1.

In addition to the post-excavation support options mentioned above, several options for pre-support are available. These include:

- Pre-grouting: used when broken ground exists that can be reached by drilling and the ground has acceptable porosity to accept grout/resin. May also be used to restrict groundwater flows.
- Piles: several types of piles exist. These may be large-diameter (>1 m) concrete-backfilled piles (with or without steel reinforcement), small-diameter concrete-filled piles, or small-diameter mini-piles, that are effectively grouted cables, bars or rods. Piles are generally installed from surface and used for poorer surface weathered conditions only.

- Pre-cable bolting: used for blocky conditions. Has limited use as can only be used if underground development access allows.

Table 1 Shaft excavation methods and corresponding ground support

Excavation method	Comments	General ground support types available
Raiseboring	Requires access to base of shaft before excavation. Generally rapid, cost-effective excavation. Non-personnel entry method.	Remotely applied fibre-reinforced shotcrete (FRS). Installed post excavation.
Blind boring	Does not require base access. Non-personnel entry method.	Sub-aqueous steel lining, grout/concrete or pre-formed concrete/reinforced concrete liners. Generally installed post excavation.
Blind sinking	Does not require access to base of shaft, personnel entry required.	FRS, mesh, rockbolts/cables, slip-form concrete lining. Temporary support installed as excavated and final liner installed at a lag distance behind face.
Strip-and-line sinking	Requires access to base of shaft for initial disposal hole excavation. Personnel entry method.	As for blind sinking above.
Hori-diam stripping	Requires access to the base of the shaft to excavate, personnel entry to pilot hole.	Blastholes over-drilled to install cables or bolts into wall before blasting.
Vertical shaft sinking machine (VSM)	Does not require access to base of shaft. From surface only. Non-personnel entry method.	As for blind sinking. More common to install support as excavated.
Alimak raising	Personnel entry, excavated from base up. Requires working constantly under exposed backs, only for small-diameter shafts.	Short length bolts, mesh. Installed as excavated to allow personnel entry.

Typically, in Australia a geotechnical investigation drillhole is drilled along the approximate centreline of the shaft alignment from which stability assessments can be made. Drillhole deviation over the length of the hole can result in information being obtained some distance from the planned shaft barrel and may reduce confidence in drillhole information representing actual shaft location conditions. Results of these stability assessments provide an indication on the need for ground support; if support is deemed necessary, the obtained geotechnical information is used to assess the support requirements. A selection of methods of ground support assessment using geotechnical data and typical available support options is detailed in the following sections.

2 Near-surface/collar shaft support

For most shafts that are excavated from the surface, a weathered profile is often present in the upper section. The thickness and rock mass characteristics of the weathered profile will vary between shafts depending on the surrounding geological conditions. In general, highly weathered/residual soils are of poorer quality than fresh rocks. Excavations through these materials are commonly not stable, when unsupported, for the required shaft diameters. The risk of shaft failure is often greatest in near-surface, highly weathered materials. Approaches to the excavation of shaft sections in weathered material, in particular the shaft collar, include:

- Pre/post sink: generally involves blind sinking and in-cycle support of the initial section of shaft. This may be done before or after excavating the main section of shaft. May also be excavated using other non-entry methods such as large-diameter piling rigs, blind bore etc.
- Pre-support: may be in the form of piling, mini-piles, pre-grouting or spiling.
- Rapid remotely applied FRS post excavation (in the case of raiseboring).

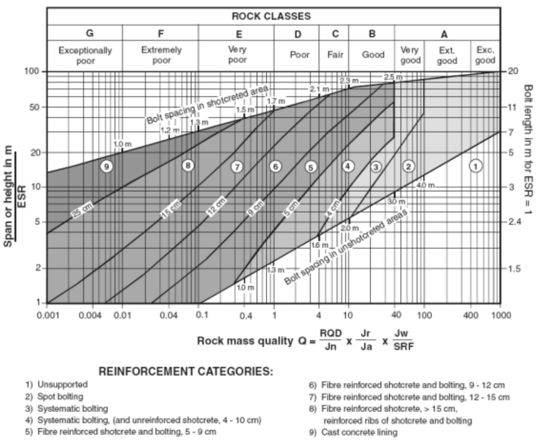
Each form of excavation and selected ground support assessment approaches is discussed in the following sections.

2.1 Pre/post-sink ground support

The pre/post sinking of the upper section of a shaft in poor ground allows for the in-cycle installation of support to manage the ground conditions. Shaft excavation can be undertaken by entry methods such as blind sinking or strip-and-line from a headframe over the shaft. These methods require both temporary support (to allow personnel access) and permanent support (for the life of the shaft). While described in a pre/post-sink context as follows (for the upper section of the shaft), the methods described may also be applied to full shaft lengths.

2.1.1 Temporary ground support

Temporary ground support is required to minimise ground movement and provide protection to personnel working in the shaft. It generally consists of bolting and mesh/FRS. The design of required temporary support can be conducted using the design charts suggested by Grimstad & Barton (1993); see Figure 1b. This involves the determination of rock mass rating Q values, applying a wall orientation factor where appropriate and applying an excavation support ratio (ESR, generally three for temporary openings). In areas where no or thinner FRS thicknesses are suggested from the design chart, mesh is often used as a surface support substitute. An example of using bolts, FRS and mesh as temporary shaft support is presented in Figure 1a. It should be noted that Q values may not be reliable in weak rock and other load/support capacity requirement methods such as those suggested in Section 2.1.2 may be required instead.



(a) (b)

Figure 1 Shaft temporary support. (a) Example; (b) Support design chart (Grimstad & Barton 1993)

Bolt length can be determined from the graph provided by Grimstad & Barton (1993) or Schach et al. (1979).

Routine geotechnical inspections of the exposed shaft walls are required to re-inform the initial geotechnical assessment used to design the ground support and to adjust the temporary support to the conditions encountered.

2.1.2 *Permanent ground support*

Permanent shaft ground support is typically a concrete/FRS or steel lining. Where blind boring excavation methods are employed, a composite liner is commonly used. Liners can either be hydrostatic (able to withstand groundwater pressures) or leaking (allow groundwater flow from outside to the inside of the shaft liner). The liner type and thickness are designed to support anticipated ground and groundwater pressures. Anticipated support loads can be determined through various methods, dependent on the type of material being assessed. In addition to numerical modelling, a selection of load determination methods is presented:

- Clay material: earth loads in clay material can be determined using the method suggested by Hutchinson (1996). This involves the use of undrained shear strength (S_u), which can be determined in the field from pocket penetrometer tests on clay material, or laboratory material testing. A more detailed approach may include the method suggested by Wong & Kaiser (1988), which involves assessment of yield zones and relates support pressures to determined yield values.
- Non-cohesive material: lateral loads can be assessed from active earth pressures as suggested by Cheng & Hu (2005) for given shaft geometries and friction angles of material. The method suggested by Wong & Kaiser (1988) can also be applied to non-cohesive material.
- Weak rock: empirical support pressure determination. This includes the determination of rock mass rating, Q values and applying empirical support pressure formulae (e.g. Barton et al. 1974):

$$P = \frac{2Jn^{1/2}Q^{-1/3}}{3Jr} \tag{1}$$

where:

P = support pressure (kg/cm^2).

Jn = joint set number rating.

Q = rock mass rating Q value, for wall values multiplied by 5 for Q > 10 or by 2.5 for 0.1 < Q < 10.

Jr = joint roughness rating.

- More competent rock: as suggested by Hoek et al. (1995), loads on the final liner may be determined using the convergence-confinement method. The method involves assessing load transfer due to ground convergence between the advancing shaft face and is highly dependent on the lag distance between the shaft face and the installed liner. The method proposed by Barton et al. (1974) described previously may also be used.
- Complex geometries/ground conditions: numerical modelling is required.

Final shaft support is generally a concrete or steel liner or a composite liner using both steel and concrete. Dependent on the complexity of the loading and the planned shaft geometry, assessment of required final support may be conducted in several manners as detailed in the following sections.

2.1.2.1 *Concrete liners*

Concrete liners can be installed as either poured concrete using formwork, sprayed FRS or as pre-cast elements. For circular shafts, the installation of a closed continuous concrete liner is critical to the development of hoop stresses within the liner to resist expected ground/groundwater loads. A selection of design approaches for varying levels of detail includes:

- Simple loading and geometry: Lame’s equations (Lame 1852) for thick-walled cylinders to determine hoop, radial and axial stresses. Estimates of required concrete liner thickness can then be based on concrete strength and a design Factor of Safety.
- More detailed assessment: design formulae suggested by Auld (1979). Assumes elastic behaviour of material and a limit state design approach to design Factor of Safety.

- More complex loading/geometry: numerical modelling using packages such as Rocscience RS2 or RS3, or others required.

2.1.2.2 Steel liners

Steel liners may be used as essentially formwork for concrete pours or used as the final liner member itself. Assessment methods for steel liner requirements include the following:

- Pressure vessel design code AS1210-2010 (Standards Australia 2010) allows the design of cylindrical shells subject to a uniform pressure applied from either groundwater or concrete. Assesses thin-walled cylinder buckling for the steel liner plate with and without stiffeners, based on acceptable levels of manufacturing defects (i.e. out-of-roundness). First-pass assessment method that can be used as a check of reasonableness of other methods, which permit non-uniform loading.
- Numerical modelling allows various load cases, geometries and boundary conditions (resistance to movement) to be incorporated into assessment. Example packages may include Strand7, Ansys, Abaqus.

2.1.2.3 Difficult ground conditions

Specific difficult ground conditions may require additional assessments and corresponding support recommendations. This may include the following:

- Squeezing ground conditions: these are areas of very weak ground at higher stresses. Assessment of the potential for squeezing conditions can be made using the method suggested by Hoek & Marinos (2000). The method involves the use of geological strength index (GSI) values that can be determined from logging data. Potential strain values are determined and general support approaches based on strain values are suggested.
- Excess shear stress on dominant jointing/structures: this can lead to stress-driven overbreak along dominant structures and may be time dependent. Fall-out levels may be acceptable. If not, additional support may be required to retain yielded material and maintain shaft shape tolerances. Assessment of potential excess shear failure may be conducted using numerical modelling using pervasive or ubiquitous jointing or the approach suggested by Seedsman (2022).
- High-stress environments: an indication of potential high-stress issues in shafts can be determined from the design charts presented by Edelbro et al. (2019). In addition, general support approach concepts are suggested in this same paper. Depths of anticipated stress drive fall-off can be assessed using numerical modelling methods, or empirical equations such as that suggested by Martin et al. (1999) may also be used.

2.1.3 Non-entry sinking methods

Non-entry sinking methods such as large-diameter piling rigs or blind boring may also be used to excavate a pre/post-sink. They may also be used to extended into more favourable ground conditions if required. Large-diameter piling rigs may be used to excavate ever-increasing diameters of shafts up to, and exceeding, 6 m in diameter. The support of these excavations is usually in the form of piling (installed by the same rig) prior to excavation (refer to Section 2.2) or remotely applied FRS either after excavation or as excavation progresses (refer to Section 2.5).

Blind bore sinking is generally used in ground conditions that may prove difficult to control in normal sinking. The method involves boring under water (with additives to increase density). This provides a confining pressure from the inside of the shaft that restricts water and loose ground material flow into the excavated shaft. Available ground support alternatives include grouted steel liners, thick concrete liners that are installed underwater and use a thin steel liner as formwork or pre-cast composite liners of concrete or reinforced concrete segments. The design of liners is based on expected ground loads (as described in Section 2.1.2) and groundwater loads. Liner design assessments are described in Section 2.1.2.

Due to support installation being underwater for blind boring, the careful design and emplacement of grout/cement is required to avoid material segregation. Any groundwater movements may also create segregation of concrete materials.

2.2 Piling

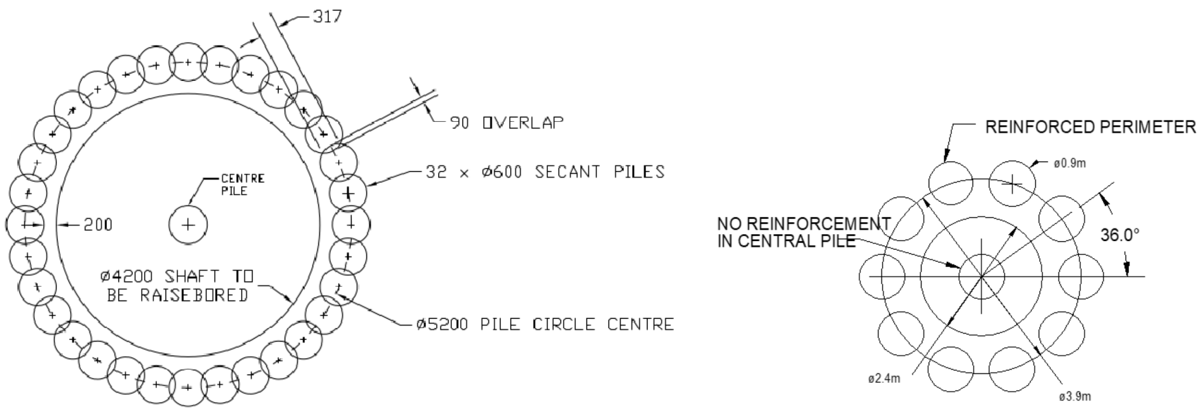
Piles are installed prior to excavation to stabilise the shaft walls as the excavation progresses. Piles are constructed by initially drilling larger-diameter holes (generally 300–1,200 mm diameter) by reaming of smaller holes with a drill rig or use of a specific piling rig, which are backfilled with concrete. The concrete is either reinforced or unreinforced. It is also common to include a combination of the two. The piles may be either overlapping (secant) or included a spacing between the outside of each pile (contiguous).

Ground conditions and the level of risk acceptance generally dictate the selection of contiguous or secant piling. If there are significant intersections of material that will dissipate or unravel between narrow spacings between piles, then secant piles are required for support. Contiguous piles can be used where the material is capable of some arching of stresses or inter-lock of material occurs.

Where raiseboring is used as an excavation option, the loads from raisebore rigs are also required to be considered in pile design. The analysis needs to consider the weight of the machine assembly and torque during operation. Large thrust loads are generated by the raisebore rig. Piles need to extend to competent material to provide some degree of end bearing capacity and resistance to bending moments.

2.2.1 Secant piling

Secant piling involves the overlap of edges of each pile. A gap is left between the planned outside of the shaft being excavated and the inside of the ring of piles. This gap is normally in the order of 200–300 mm and it provides some room for pile deviation. Material in this gap generally falls as the shaft is excavated exposing installed piles (Figure 2a). It is common to install piles in a primary/secondary sequence, with secondary piles being reinforced to some degree. The diameter of piles is generally determined by available equipment, and spatial and verticality accuracy requirements. An example layout of secant piles for a shaft is presented in Figure 2a.



(a) (b)

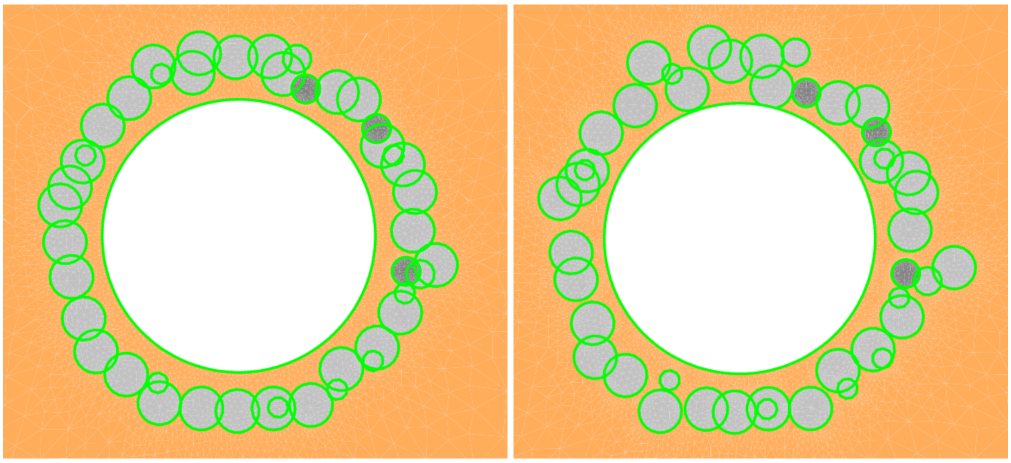
Figure 2 Example shaft piling layouts. (a) Secant piling; (b) Contiguous piling

The amount of overlap of each pile is generally determined by the required thickness of continuous concrete ring to support anticipated ground loads. In practice, larger-diameter piles are preferred as they provide greater thickness of the continuous ring of concrete and are generally easier to maintain positional accuracy when drilling.

An example of secant piles that have been installed outside a raisebore diameter and then raisebored is presented in Figure 3. To note in this figure is that the material between the raisebore and piles has fallen upon excavation, but not past the piles.

Figure 3 Example of exposed secant piles after raiseboring

It is generally easier to maintain positional and vertical accuracy for larger-diameter piles. However, most equipment cannot exceed depths of approximately 30–40 m before exceeding accuracy tolerances. Gaps in piling arrangements can develop as pile depths increase and location accuracies decrease. An example of pile deviation is presented in Figure 4. In this case, additional piles were drilled to fill the gaps in between piles that had developed at depth.



(a) (b)

Figure 4 Example shaft pile (0.52 m diameter piles) deviation for 4.2 m diameter shaft (design shown in Figure 2a). (a) Cross-section at 14 m depth; (b) Cross-section at 28 m depth. Grey piles are infill piles to bridge gaps in original pile installation

2.2.2 Contiguous piling

Contiguous piling involves larger -diameter piles that have a gap between the outer edges of each pile. An example layout is presented in Figure 2b. The determination of pile diameter is generally dictated by ground conditions, available equipment and accuracy requirements/tolerances. Generally, larger-diameter piles are preferred as they tend to be easier to drill more vertically and have greater resistance to shear and vertical loads.

Pile depth is assessed based on ground conditions that require support. The piles are not connected and cannot generate hoop stresses and, therefore, are primarily support sections of ground in shear. Resistance to bending of the slender piles is minimal and applied ground loads may cause bending failure of the piles. If large ground pressures are expected, then secant piles may be more appropriate.

The gap between the piles is determined as the distance at which the ground material will potentially either arch (for cohesive material) or be self-inter-locking (broken ground). The assessment method is dependent on the type of ground material. Methods of assessment include the following:

- Cohesive material (clays): the potential for plastic flow of material between piles can be assessed using the method suggested by Ito & Matsui (1975). The method requires estimation of material shear strength and pile diameters to determine required gap between piles.
- Broken ground: a conservative assumption can be made that broken material sits somewhat loosely in the ground. Block size (diameter) of material can most simply be determined from logging data using various inputs (joint count, RQD or block volumes) as suggested by Palmstrom (2000). The assumption can then made based on simplified particle flow rules of thumb that material will tend to block rather than flow through a gap that is less than 3–5 times (Marlow et al. 2013) the block size of material.

2.3 Mini-piles or spiling

Mini-piles or spiling are generally smaller-diameter (approximately 100–300 mm diameter) holes drilled parallel and outside the planned shaft perimeter prior to excavation. Holes can be normal or reamed drill rig holes or drilled with a small piling rig. Support installed in these holes is generally a form of grouted-in steel element. The steel element installed can be steel bar, cable, drill rods or other that may be readily available on site.

Mini-piles or spiling are designed to lock in larger blocks of material that may slide into the shaft wall. As such, they are applicable to blocky ground and not to unravelling conditions. They do this through the shear strength of the elements installed in each hole (usually the steel element). The shear strength of the installed element is designed to match the anticipated loads of blocks that may slide from sidewalls. Designs can consist of practically any size hole and installed element, providing flexibility to match available site machinery and materials.

The potential block sliding loads can be assessed through wedge assessments from known jointing/structure orientations. These orientations may be from acoustic televiewer (ATV) surveys from the shaft geotechnical drillhole or site-wide general defect orientation data.

The spacing of mini-piles is assessed to match the anticipated shear loads and prevent unravelling of material (based on block size as described in Section 2.2.2). Factors of safety applied to shear strength of material follows Standards Australia (2002, 2018, 2020).

Mini-piles or spiling are long and slender with respect to the diameter. This leads to them being susceptible to bending moment failure. As such, they are not suited to bridging a long distance between stable areas along the shaft length. Significant areas of potentially unravelling material may either fail in between the mini-piles or, if locked in behind the mini-piles over a significant length along the shaft, cause bending failure of the piles.

2.4 Pre-grouting

Pre-grouting can be undertaken in material to restrict/prevent groundwater flows, improve ground quality or both. Pre-grouting may be used with any type of excavation method; however, for sinking methods it is more common as a groundwater control method rather than ground improvement.

While very fine particle grouts/resins are available to inject, ground conditions are not always amenable to grout injection for ground improvement. Ground porosity is critical to the mechanism of grout injection that will lead to ground improvement, but also the interconnected porosity is critical. Generally, pre-grouting is used in broken, loose material with low clay filling on joints/structures as it provides a suitable medium to grout and obtain improvement of ground quality. Specialist advice, previous experience, permeability testing and in situ trials are required to determine if grouting could be an effective ground improvement solution in a given material.

Grouting is typically done in a primary/secondary (and potentially tertiary) sequence from several holes drilled outside the perimeter of the planned shaft. An example layout of pre-grouting holes for a 2.4 m diameter shaft is presented in Figure 5.

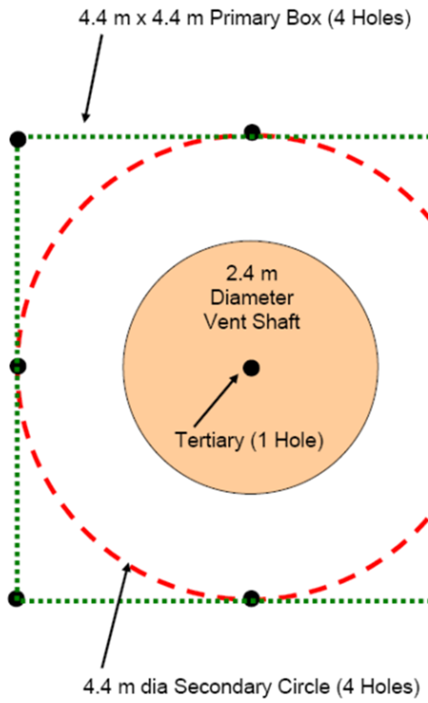


Figure 5 Example shaft pre-grouting hole layout

The use of core drilling for secondary and tertiary holes allows an assessment of grout migration distances and locations. A weak phenolphthalein solution can be used in non-calcareous rocks to locate cementitious grouts in recovered core. Additional holes may be required if the desired spread of grout has not been achieved.

2.5 Rapid support post excavation

The rapid support of shafts, post excavation, is conducted by lowering and grouting in a steel liner or, more commonly, by the application of remotely sprayed FRS. The use of these types of support is dependent on the potential stand-up time of the material that is being excavated. Stand-up times for raisebored shafts can be assessed using the method suggested by Coombes et al. (2011). The method follows that of Bieniawski (1989), where rock mass rating (RMR) is assessed and plotted against span to graphically assess stand-up time. Coombes et al. (2011) adopted the method for raisebores with case studies. The assessment chart is presented in Figure 6. Further work and back-analysis are required to provide confidence in the chart.

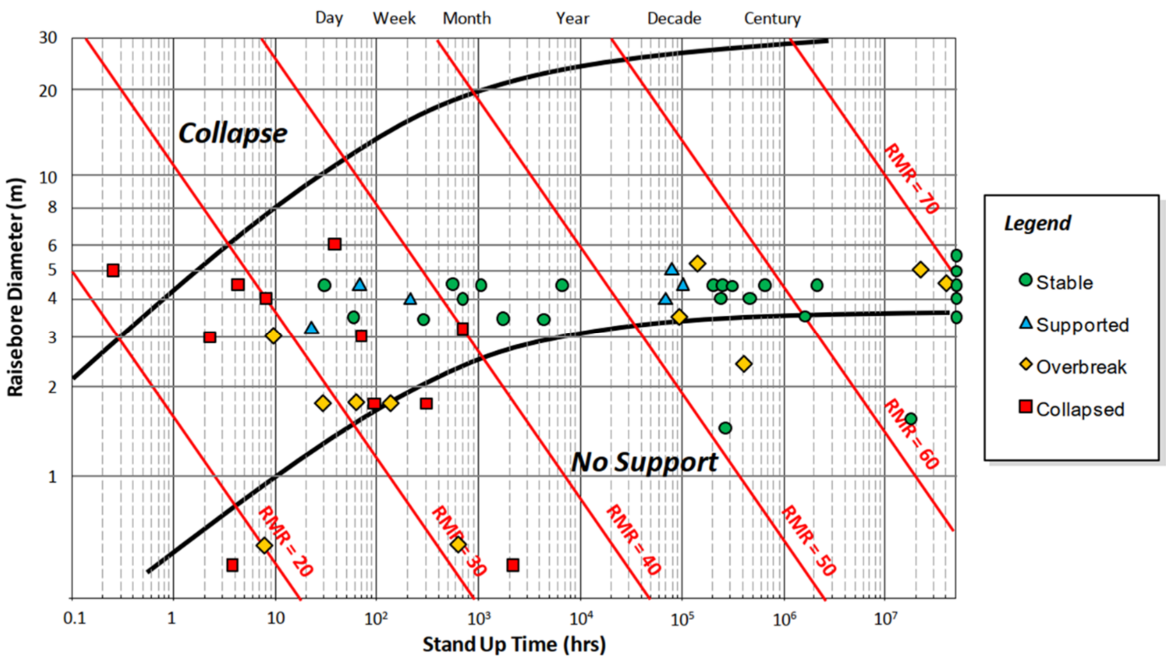


Figure 6 Stand-up time assessment chart (Coombes et al. 2011)

The stability of ground material directly below the surface collar of a shaft is critical. Construction works generally require the access of personnel and large equipment up to, in and around shaft collars, and any failures may have large consequences. The assessment of very short stand-up times for near-surface sections of shafts can, however, be difficult and large variations may be expected between assessment results and actual performance.

The reliance on a material's short-term stability before installing support is generally a risk-based decision. The major risk is collapse connecting to surface endangering personnel and machinery, and requiring significant recovery works or shaft abandonment. In addition to formal assessments, guides that may assist in anecdotally assessing near-surface shaft stability include steep surface excavations, previous shaft experience and drillhole/pilot hole fall-out.

The design of steel liners has been considered in Section 2.1.2.2. The design of FRS thickness can be determined from the shear strength of the FRS, the area of resistance and anticipated support pressure requirements. It should be noted that installing fibrecrete remotely can introduce installation quality issues, particularly in overbroken areas, and allowance for installation issues should be made.

3 Support of broken areas or poor ground

The support of broken areas or poor ground that are otherwise in generally favourable conditions, may be undertaken using the following methods:

- Pre-grouting.
- Cablebolting from remote location.
- Rapid support post excavation.

3.1 Pre-grouting

Methods of pre-grouting of isolated sections of potentially unstable or groundwater flow prone areas outside the pre/post-sink area are undertaken similarly to that described in Section 2.4. Dependent on the location of poorer/water-bearing zones and available access for drilling, drilling of grout holes may be simply an extension of pre-sink area holes (parallel to the shaft but extended to depth) or drilled from a more convenient access (e.g. underground development).

Drillhole accuracies can limit the effectiveness of grouting programs, and locating drillhole collars as close as possible to the area to be grouted is advantageous.

3.2 Cablebolting from remote location

If isolated areas of generally blocky ground are determined to require support, they may potentially be supported by grouted cables if drill access permits. These may be parallel to the shaft (as for mini-piles/spiling as described in Section 2.3) or drilled oblique to the shaft from other underground development.

Drilling parallel to a shaft to install cables will generally result in loads (typically shear loads) on the cables as blocks in the shaft walls move. While cables do provide some load capacity in shear, they are more efficient in tension and may not resist required loads. Drilling accuracy is critical in the case of raiseboring, as raisebore heads may tangle in cable bolts if they intersect the planned shaft barrel. Fibreglass cables can be considered if there is a risk in cables intersecting the raisebore face area.

Drilling cables from a remote underground location will be dependent on the geometry of the available underground locations. Unless various accesses are available, support will be drilled across the shaft walls in one direction, and if block movement occurs, cables would be loaded in various combinations of shear and tension, which may not be the most effective use of cables. Cables are most likely to be drilled prior to shaft excavation, in which case, cables cannot be installed across the shaft barrel in the case of raiseboring (as raiseboring heads would tangle rather than cut cable bolts). This leads to a ‘shadow’ in support coverage on the portion of the shaft wall opposite the side from which cables are installed.

3.3 Rapid support post excavation

Isolated areas of poorer ground can be supported rapidly after the excavation is completed. The stand-up time of the poor ground zones may allow sufficient time to access the zones in the shaft. Assessment of stand-up times and support elements is described in Section 2.5 for pre-sink areas and would be the same for remote areas away from the pre-sink area. However, in the case of raiseboring, the required stand-up times are longer for areas deeper in the shaft.

Fall-off from areas that are further from the collar area can be more tolerant of additional fall-off, given that they will not affect surface/collar stability. A longer time between excavation and support installation could be acceptable. However, in the case of remotely applied FRS, it should be noted that excessive wall fall-off can create difficulties in support application. This may lead to significantly more FRS volumes being required to be applied and potentially a decrease in support installation quality.

4 Brow support at base of shaft

In the case of development at the base of the shaft being mined before excavation (raiseboring, strip-and-line, hori-diam and Alimak methods), brow ground support can be installed before shaft excavation. The brow of a shaft is stress relieved on at least two sides and will generally have low confining stresses. This can lead to falling of any defined blocks or unravelling of loose areas. Any failure at the brow can propagate upwards and progressively undercut the shaft sidewalls.

The installation of tight bolting patterns, high-capacity surface support and long cable bolt support is typical for shaft brow positions. The amount of support can be determined from wedge assessments from mapping of shaft base development. It is critical that cable support is installed only outside the planned shaft barrel in the case of raiseboring as raisebore heads can tangle around cable bolts.

5 Discussion and conclusion

Shaft ground support options are generally dependent on excavation method and available access (personnel entry or not); a single shaft may have a combination of methods. While many methods of assessing shaft behaviour and support requirements exist, typical shaft ground support types and a selection of load capacity/ground support requirement assessment methods have been described in this paper.

Many of the methods are adapted from tunnelling or civil surface assessments but are assessing the same mechanisms that present in what is effectively a tall wall of a shaft. However, non-shaft-specific methods may provide conservative results due to the added stability effect of a circular shaft shape.

The selection of appropriate load/stability/support method for each situation is dependent on the anticipated failure mechanism of the ground material and excavation method:

- Will the material behave similar to a soil or a rock?
- Unravelling failure is likely to require complete surface coverage (FRS, concrete, steel liner etc.).
- If stress spalling is likely, will it extend to a depth that will critically affect the shaft performance?
- If support is installed as excavated (sink and line methods), then assessment methods that account for wall closure due to excavation will be required.

In areas where a mixture of ground conditions is encountered, several methods may be applied to the same material and an engineering judgment made on the appropriate results or range of results to adopt. This may be the most conservative result, but not necessarily.

Many of the selected methods are either empirical or closed form solution approaches and provide a good general assessment of potential shaft/support performance. Advances in availability of structural data (ATV surveys) and accessibility to 3D numerical modelling packages that can process larger amounts of data allow more detailed modelling of shaft stability/support. These models will become more useful as application and calibration of these models progresses. It is suggested that a variety of methods of assessment are considered for each situation to develop a pragmatic solution to ground support designs.

It is important to conduct back-analysis of all failures in shafts to increase our understanding of material behaviour and the types of failure mechanism. This also allows calibration of numerical models and comparison to empirical predictions.

References

Auld, FA 1979, ‘Design of concrete shaft linings’, *Proceedings of the Institution of Civil Engineers*, vol. 67, no. 2, pp. 817–832.

Barton, N, Lien, R & Lunde, J 1974, ‘Engineering classification of rock masses for the design of tunnel support’, *Rock Mechanics*, vol. 6, no. 4, pp. 189–236.

Bieniawski, ZT 1989, *Engineering Rock Mass Classification*, Wiley, Hoboken.

Cheng, YM, & Hu, YY 2005, ‘Active earth pressure on circular shaft lining obtained by simplified slip line solution with general tangential stress coefficient’, *Chinese Journal of Geotechnical Engineering*, vol. 27, no. 1, pp. 110–111.

Coombes, BM, Lee, MF & Peck, WA 2011, ‘Improving raisebore stability assessments and risk’, *Proceedings of the 2011 Canadian Institute of Mining, Metallurgy and Petroleum Conference and Exhibition*, Canadian Institute of Mining, Metallurgy and Petroleum, Westmount.

Edelbro, C, Brummer, R, Pierce, M, Sandstrom, D & Sjoberg, J 2019, ‘Raiseboring in difficult rock conditions’, in J Hadjigeorgiou & M Hudyma (eds), *Ground Support 2019: Proceedings of the Ninth International Symposium on Ground Support in Mining and Underground Construction*, Australian Centre for Geomechanics, Perth, pp. 185–198, https://doi.org/10.36487/ACG_rep/1925_11_Edelbro

Grimstad, E & Barton, N 1993, ‘Updating of the Q-System for NMT’, *Proceedings of the International Symposium on Sprayed Concrete*, Norwegian Concrete Association, Oslo, pp. 46–66.

Hoek, E, Kaiser, PK & Bawden, WF 1995, *Support of Underground Excavations in Hard Rock*, A.A. Balkema, Rotterdam

Hoek, E & Marinos, P 2000, ‘Predicting tunnel squeezing’, *Tunnels and Tunnelling International*. *Part 1 – November 2000*.

Hutchinson, DM 1996, *Handbook on Liner Design for Drilled Ventilation and Access Shafts*.

Ito, T & Matsui, T 1975, ‘Methods to estimate lateral force acting on stabilizing piles’, *Soils and Foundations*, vol. 15, no. 4.

Lame, G 1852, *Lecons sur la Theorie de l Elasticite*, Gauthier-Villars, Paris.

Marlow, P, Webber, S, Mikula, PA & Lee, M 2013, 'Bored reinforced piles for raisebore support: four case studies, and guidelines developed from lessons learnt', *Mining Technology*, vol. 122, no. 3, pp. 159–165.

Martin, CD, Kaiser, PK & McCreath, DR, 1999, 'Hoek-Brown parameters for predicting the depth of brittle failure around tunnels', *Canadian Geotechnical Journal*, vol. 36, no. 1, pp. 136–151.

Palmstrom, A 2000, 'Block size and block size distribution', paper presented at GeoEng 2000, Melbourne.

Schach, R, Garshot, K & Heltzen, AM 1979, *Rock Bolting – A Practical Handbook*, Pergamon Press, New York.

Seedsman, RW 2022, 'Options for designing a hold-retain strategy for excavations in stratified rock', in RE Hammah, TE Yacoub, A McQuillan & J Curran (eds), *The Evolution of Geotech – 25 Years of Innovation*, Routledge, Milton Park, pp. 569–575.

Standards Australia 2002, *Structural Design Actions, Part 0: General Practices (AS1170*), Standards Australia, Sydney.

Standards Australia 2010, *Pressure Vessels (AS1210-2010)*, Standards Australia, Sydney.

Standards Australia 2018*, Concrete Structures (AS3600),* Standards Australia, Sydney.

Standards Australia 2020, *Steel Structures (AS4100)*, Standards Australia, Sydney.

Wong, RCK & Kaiser, PK 1988, 'Design and performance evaluation of vertical shaft: rational shaft design method and verification of design method', *Canadian Geotechnical Journal,* vol. 25, pp. 320–337.