Towards automated tunnel traversing for the control of deep level mining

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Abstract

Huge resources of gold and platinum remain at depths uneconomical to mine using conventional methods. This paper attempts to make a contribution towards the surveying control necessary for a fully automated mining operation controlled from the surface.

Tunnelling projects relying on multiple traverse lines with frequent gyro azimuth checks point to the dangers of atmospheric refraction to any form of electromagnetic measurement, particularly pertinent to the stratified air in a hot deep level mine with cooling ventilation. The tunnel for the Large Hadron collider at Cern, Switzerland, seeking sub-millimetre alignment of its collimators over 27 km introduced invar wires (steel alloy with a very low thermal expansion co-efficient) and short range photogrammetric methods for measuring distances and connecting to pegs to overcome the errors caused by refraction.

Using conventional equipment initially, a new method of traversing is proposed connecting “reference stations” with forced centred traverse segments measured by an imaging total station. While the scope of the survey neither allows for the avoidance of refraction nor attempts to measure refractive errors, it does pave the way for automation. The measuring instruments can be substituted in future.

The over determination of the reference station (comprising six points) and double traverse links between stations inherent in the method allow for parametric network adjustment of the survey. This approach produces a standard deviation of ±25 mm over a 215 m eight leg-traverse, comparable with existing manual methods. The work reported in this paper is part of an MSc research study in the School of Mining Engineering at the University of the Witwatersrand.

Introduction

Atmospheric refraction in tunnelling

Surveying practice always distinguishes between angles measured in the horizontal plane from those measured in the vertical principally because the first is referenced directly or indirectly to the north/south direction and the second to the plumb line or direction of gravity. This is not the only distinction as it is generally acknowledged that calculations based on vertical angles produce poorer results and this is due to the path of the line of sight following a curve as it passes through layers of the atmosphere of different densities above the terrain. It is known as horizontal refraction and over longer lengths is corrected for assuming average conditions.

Horizontals can be similarly affected if the line of sight grazes an obstacle and these measurements are therefore avoided. The method of levelling is used principally to eliminate refraction in the vertical plane. The last point to note is that any distance measured electronically gives the curve length referred to above rather than the straight line distance and, since it is based on the speed of light, this also varies with density of the air. Corrections are applied once again based on average conditions. Surveys of the utmost precision use a taught wire for distance measurement together with close range photogrammetric means to register the ends of the wire with survey reference points achieving sub-millimetre precision, free of refractive concerns.

In the particular case of mine tunnelling where portals to the surface for position checks are not a possibility, what sort of survey precision is required? Unlike the conditions on surface described above, temperature variation between the rock face and the centre flow of ventilation can be extreme producing a highly stratified density gradient throughout the tunnel affecting horizontals as much as verticals and also distances due to the close proximity of the line of sight to the rock face. Of course the effect on a single typically short traverse leg would be negligible and as long as these errors were random, as would be the case if the line of sight followed the centre of airflow, they would compensate perfectly. Consider a curve in the tunnel or in the worst case a spiral, a traverse close to one sidewall, all pegs only in the hanging, sidewall or footwall, these would all produce systematic errors accumulating along the length of the tunnel.

The classic and vital solution to these problems is of course levelling which, although not entirely free of refraction, gives an independent height determination and secondly the frequent gyro correction necessary to maintaining bearing.
**Sight length**

The length of the backsight often constrained by curves and obstructions has a direct effect on preservation of bearing and is usually only corrected by the check survey or gyro base. A better way of maintaining orientation is required.

**Stationing**

Bearing in mind the objective of automation, a more flexible method of determining the position of the measuring instrument inside the tunnel is required. Is it possible to replace traditional pegs or reflectors and use the unique section of the tunnel at various intervals for fitting purposes as can be done for point cloud registration or a section of the wall for a photogrammetric fix? Would it be possible to contour a section of the rockface and use the high points or even the rockbolts as pegs? These distinct possibilities will be covered in the literature review below. At this point it is important to state that one of these methods is assumed to be practical and is simulated by station adjustment to an array of six small reflectors placed around the tunnel or on one face of the tunnel. As it turns out, this manual method (described later under the heading “Signalisation”) provides an excellent solution, positioning the instrument to 1 mm.

**Virtual traverse**

A new method of traversing is evaluated which virtually eliminates centring errors, greatly reduces orientation loss due to short backsights, takes advantage of high precision station fixes, is well suited to automation and provides the redundancy necessary for estimating the precision of the advancing survey. It is described under the heading below and is the basis of the survey measurements and subsequent computation.

School of Mining Engineering at the University of the Witwatersrand has recently completed a mock up tunnel in the basement of the school, which provides a framework for the trial survey. A Topcon 5” imaging total station is used for measurements. It can be remotely controlled, projecting the telescope image onto a computer screen for manual or automated sighting and, produces reflector and reflector-less short distances to the same precision of 1 mm. Columbus network adjustment software by Best-Fit Computing is chosen for evaluating the results. These current parameters, and together with the automated gyro attachment available for this instrument represent the same options currently available to most mining operations for control surveys underground.

**Literature review**

The only way forward at this early stage is to simulate some of the surveying ideas developed by teams at Oxford and Hamburg universities in preparation for the alignment of the Large Hadron collider (Prenting and Green, 2004) since these have had the greatest influence on the methods described in this paper. To explain the methods better, Fig. 1 illustrates the simplified concept of the geodetic reference survey in 2D. The measurement bar or Invar wire or ‘laser within a vacuum tube’ is mounted on a train carriage (Fig. 2) and moves between sidewall reference targets. Short perpendicular offsets shown in Fig. 1 are measured by “photogrammetric means” (as shown in Fig. 3) and the overlapping readings allow for computation and checking of the target co-ordinates.

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**Fig. 1: Concept of geodetic reference survey simplified to 2D (Prenting and Green, 2004).**
Fig. 2: Measuring train on the tunnel wall (Prenting and Green, 2004).

Fig. 3: Optical 2D sensor (Prenting and Green, 2004).

Fig. 3, as with the rest of this description, presents the 3D survey method in highly simplified form. The “photogrammetric fix” provides an optical connection to the target over shortest possible length in order to minimise refraction.

Although this is more of a monitoring survey in an existing tunnel than an advance of survey control, it serves as merely a seed for the idea of the virtual traverse. Take for example the optical range-finding arrangement in Fig. 3. Two more appropriate solutions are found, well-expressed and quantified in the work of H Reuter (Reuter, 2012).

Fig. 4: Laser scanner set up positions during track-way survey (Reuter, 2012).
Both the laser scan in Fig. 5 solution and the photogrammetric one in Fig. 6 are able to position each footprint to within 1 mm, shown in Fig. 6.

Nor is this the exact solution sought. Given the known position of the footprints, what is the position of the terrestrial cameras or scanners? This is merely the inverse problem. The ranges from instrument to footprints are very similar to those in the reference stations of this paper.

The seminal study of tunnelling refraction involved the so-called Chunnel tunnel developed from England and France to meet midway beneath the sea. The attempts to systematise the refractive errors involved running the start of the traverse inconveniently along the centreline since this is where all the equipment is found and then in a zig-zag fashion between the wall-mounted brackets (Koritkke, 1990).
Centerline traverse


Zig-zag traverse

Table 1: Eurotunnel verification survey.

A more recent railway tunnel in Spain (Fig. 7) showing the expansion of error ellipses with chainage produced by Least Squares adjustment of the survey (Velasco, 2010).

Fig. 7: Comparison of error ellipses with and without gyrotheodolite observations adjustment (Velasco, 2010).
Signalisation

Is it possible to replace traditional pegs and use the unique section of the tunnel for fitting purposes such as point cloud registration of a profile of the wall or a photogrammetric fix? Would it be possible to contour a section of the rockface and use the high points or even the rockbolts as pegs as shown in Fig. 8. In order to simulate this, small pieces (<10 mm) of chevron adhesive tape as used for roadside signage is found to be a very good reflector for both measuring modes of the total station and is effective to a range of 8 m. This is shown in the smartphone picture taken with flash (Fig. 9). The reflectors are in general placed anywhere around the section of the tunnel representing the keypoints described earlier, but in this case minimum focus distance allowed for only one wall in the small tunnel.

Fig. 8: Working face (Business Day, 2014).

Fig. 9: Reference Station 1A.

Fig. 10: Laser illumination and background illumination.

Telescope images of the reflector tape illuminated by laser beam and then ambient tunnel lighting showing the possibility of sighting with millimetre precision in Fig. 10. It is less obvious to which point the distance is measured. Small prisms of the same diameter would increase the distance precision and be more effective for auto pointing.
If this all sounds rather speculative, the results shown in Fig. 11 indicates that the method works. The six points A1 to A6 of reference station 1A (Fig. 5) are fixed by polar from a baseline B1 – B2. A new station TR is then surveyed to within 2.5 mm in X, Y and Z at 95% confidence level.

![Fig. 11: Station fix and new point solution.](image1)

**Reference station**

The four reference stations of the control traverse are shown in Fig. 12. The bottom right is the station described later. Target points are chosen to provide good intersection angles for fixing in 3D from a traverse baseline (upper left) and the position of future set-ups is similarly chosen with regard to a good fix (upper right). This method provides equally good control of level as horizontal position.

![Fig. 12: Control traverse.](image2)
Virtual traverse

In order to overcome the loss of orientation due to a short back-sight (conventional traverse through points T1, T2, T3 and forward), traverse segments between reference stations R1, R2 and forward where the bend-point is a temporary position held by the instrument and reflector, a force-centred position to a common tri-brach is proposed. In this way centring errors are virtually eliminated at the three occupied positions and the orientation taken forward is effectively between the reference stations R1 and R2 of the virtual traverse. Fig. 13 shows the difference between Conventional (black line) and Virtual traverse (red line).

Since mining advances in stages, the whole segment R1 – R2 will have to be re-observed merely to pick up bearing for the next station R3 (and any alignment, and setting out required). This is not seen as a disadvantage as the additional work merely serves as the check survey or more importantly provides the degrees of freedom essential to network adjustment of all the measurements.

Considering the primary purpose of this paper is automation, two survey robots (comprising imaging stations and gyro) would be able to conduct this survey in leap-frog manner, merely extending tripod legs beyond their tracks for stability (similar to a moon buggy).

A Priori - personal equation

An essential input to network adjustment is estimation of the precision of measurements and centring. The usual method when measuring over large distances on surface is to pit yourself together with the instrument under prevailing weather conditions against a series of overlapping observations and check for consistency. This is itself a parametric adjustment which produces the required standard deviations, known as a priori since they are independent of the control traverse. Fig. 14 shows an overdetermined set of angle and distance measurements.

Fig. 13: Conventional (black line) vs virtual traverse segments (red line).

Fig. 14: Overdetermined angle and distance measurements.
**a Posteriori standard deviation**

The double measured closed control traverse (Fig. 15) through reference stations 1A, 2A, 3A, 4A, 1A with 1A held fixed and external orientation to E1 provides the framework for further testing. Over the reference station distances such as Y2 to 2A2, 5” subtends 0,0002 m and over a typical traverse leg such as T32 to T42, 0,001 m. Although this seems improbable, the instrument nevertheless invariably measured directions and verticals to this 5” precision in the test tunnel. Furthermore the distances agreed to 0,001 to 0,002 m.

![Fig. 15: Closed traverse, station 2A detailed.](image)

Using a Priori inputs initially of 5” for horizontals and verticals, 0,001 m for distances and 0,000 m for centering not surprisingly produced much larger observation standard deviations (a Posteriori) after the adjustment of the closed traverse. Clearly the instrument precision of 5” far exceeds the requirements of the survey and most of the errors arise from the signalisation.

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<th>Stations</th>
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<td>170.0000</td>
<td>AT T2</td>
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</table>

**Table 2: Measured and adjusted with a Priori and a Posteriori standard deviations.**

In order to bring a Priori and a Posteriori standard deviations into better agreement, after much trial and error based on Fig. 15 and Table 2, the directions and verticals to reference targets are assigned 170” and to traverse stations 10”. All distances to targets are assigned 0,0030 m and to traverse stations 0,0015 m. Table 2 shows sample of two measured and adjusted target direction observations and two measured and adjusted traverse direction observations together with the new updated a Priori and a Posteriori standard deviations.

**Test framework results**

Putting all this together produces reference target points within error spheres of 2 mm radius at the 66% confidence level (Table 4). Interestingly the vertical precision is identical to the horizontal. It appears that the traverse legs are solely responsible for this error, and the reference point itself is virtually error free. Fig. 7 confirms this.

2A1 to 2A6 of similar precision constitute reference station 2A. Point X2 of slightly higher precision was derived from these points.
3A1 and X3 are representative of reference station 3A.

4A1 and X4 are representative of reference station 4A.

It should not be forgotten that all stations are temporary forced centred positions, “virtual pegs”, except the actual targets 1A*, 2A*, 3A*, 4A*. Table 3 shows the ratio between a Priori and a Posteriori variances which should be close to 1.

![Table 3: Agreement between variances.](image)

Table 4: Typical results of station targets and traverse stations.

Open traverse

Removing the tie in the centre of the traverse and closing on new points ZZZ and XXX and comparing to ZZ and XX which are tied to station targets 1A* provides the spatial misclosure as shown in Table 5.

![Table 5: Misclosure.](image)
This is about twice the predicted size of the error ellipsoid shown in Table 6.

\[
\begin{array}{cccc}
\text{AT-} & \text{Semi-Major (2D error chi-sq)} & 1.832, \text{ cons } 0.005 & \text{Semi-Medium} \\
XX & 0.0042 & 0.0028 & XZ \\
XX & 0.0001 & 0.0075 & ZZ \\
ZZ & 0.0002 & 0.0006 & XZ \\
ZZ & 0.0002 & 0.0075 & ZZ \\
\end{array}
\]

*Table 6: Error ellipsoids.*

Graphically, the growth of the error ellipsoids along the traverse can be seen in Fig. 16, the vertical bar indicating the elevation component.

**Open traverse discussion**

Although data is processed by 3D network adjustment, the only possible survey method through underground tunnels is the traverse with its familiar strengths and weaknesses, in this case mostly doubly measured and with no centring errors. The classic problem of a circular traverse is hidden systematic distance errors which play in our favour here since it is eliminated from all comparisons. That leaves traverse distances with standard deviation of 0.0015 m and directions of 10”.

The average leg of 31 m will have a length precision of ±0.0015 m and a directional precision of 10” which subtends a surprising ±0.0015 m at right angles and similarly in elevation, producing a sphere of error.

![Fig. 16: Open traverse.](image)

In the worst case, considering the 0.025 m misclosure caused by horizontal directions alone which translates to a 24” loss of bearing over the 215 m traverse. Supposing reference station 1A, T1, T2, T3, T4, reference station 3A, T5 and T6 represent the eight setups of a conventional traverse, then this equates to a 3” cumulative loss per setup.

Theoretically, using well known geodetic surveying practice, a 5” instrument should produce an angle, the difference between two directions, with a standard deviation of \(\frac{5”}{\sqrt{2}} = 7”\) (since variances are additive, standard deviations accumulate according to the root). A traverse, double measured puts the average angle measured at 7”/ \(\sqrt{2} = 5”\) and the expected standard deviation of the cumulative angle or bearing over the whole traverse comprising 8 legs at 5”/ \(\sqrt{6} = 14”\) which is half of that actually achieved.

Fortunately these pocket statistics with all their assumptions agree with Table 5, in that the method presented should produce an average misclosure of 0.0142 m.
Conclusions

The new method of underground control surveying developed in this paper has been thoroughly tested and
performs to the precision achieved. No measurements were discarded and the section from reference station 2A
to 4A was triple traversed because of some doubts while underground. The theoretical precision should be held
in doubt until shown otherwise. It is in any case unlikely that conditions in an actual mine would produce a
superior result.

One of the satisfactions of this study was the performance of the adjustment software, the theoretical
foundations decades old and yet little used underground, which as always proves its worth to the surveyor as a
personal quality control.

Although just a simulation the signalisation together with the concept of the reference station works so well that
it may find application in current working mines with little modification. It was noticed that not all total stations
measure successfully to these targets.

There is plenty of room for further development. All measuring devices should be replaced with temperature
and pressure invariant alternatives and perhaps quantum direction control will become a reality. The control
survey should remain distinct from mass data acquisition instruments so that this part of the surveying task can
be optimised for precision alone.

References

East London Polytechnic, (1983)
high speed railway in spain,” FIG working week 2010, Spain.
alignment of large linear accelerators” Proceedings of LINAC 2004 (p. 690), Lubeck, Germany:
and Mapping Conference and Exhibition, Warwick, 1989
site” Retrieved 10 June, 2014, from africageoproceedings.org.za: africageoproceedings.org.za/wp-
content/uploads/2014/06/96_Ruther_Bhurtha_Schroeder_wessels.pdf

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