Forensic Engineering
Volume 167 Issue FE3
Negotiation of poor ground in an undersea tunnel
Coggan and Yelland

ice | proceedings

Proceedings of the Institution of Civil Engineers
Forensic Engineering 167 August 2014 Issue FE3
Pages 143–150 http://dx.doi.org/10.1680/feng.13.00027
Paper 1300027
Received 31/10/2013 Accepted 06/12/2013
Published online 04/02/2014
Keywords: excavation/tunnels & tunnelling

ice
Institution of Civil Engineers
publishing

Negotiation of poor ground in an undersea tunnel

John Coggan MSc, PhD, CEng, FIMMM
Associate Professor of Geotechnical Engineering, University of Exeter, UK

Scott Yelland MSc, CEng, MSCM, FIMMM
Chief Operating Officer, Forte Energy, London, UK

The 2·65 km Gwithian outfall tunnel formed part of the tunnelling operations under the 'Clean Sweep' sewage/sewerage distribution system upgrade within the south-west region of the UK during the 1990s. The 3·0 m high by 2·8 m wide, rectangular, tracked tunnel was constructed by Trafalgar House Construction using drill and blast techniques to intercept a series of pre-drilled diffuser units 25 m below the seabed in St Ives Bay. In view of the close proximity to the seabed, and the risk of water ingress, systematic probe drilling was performed at regular intervals during construction. Where necessary, in order to reduce the water-make to within pre-defined limits, cementitious grouting of the ground ahead of the advancing face was undertaken. One major fault zone required extensive grouting, as initial probe holes were making in the order of 200 gallons per minute (909 l/min). Tunnel advance through 'exceptionally poor' ground required modification to the excavation methodology and implementation of additional support measures. Evaluation of geotechnical data from the undersea tunnel suggests that the Q-system provided a sound basis for assessment of rock quality and for guidance on associated support requirements. Good correlation was obtained between mapped Q-values and tunnel advance rates. Importantly, engineering judgement informed final support recommendations.

1. Introduction

The 2·65 km long Gwithian outfall tunnel was driven by Trafalgar House Construction as part of the Penzance and St Ives sewerage and sewage treatment scheme. Figure 1 shows the location of Gwithian, on the north coast of Cornwall, UK. The tunnel was driven by drill and blast techniques using a rail-mounted 'twin-boom' Tamrock electro-hydraulic rig for drilling operations. Blasted material was removed from the face using an Atlas Copco Hägglund 8HR2 shuttle train and gathering-arm loader. The access shaft was 60 m deep, with a finished diameter of 5·7 m. The tunnel was constructed to house a 900 mm outfall pipe, driven at a gradient of plus 1:400 with a final target 25 m below the seabed diffuser location in St Ives Bay. After commissioning, the tunnel was allowed to flood.

Rock mass classification, based on the Q-system (Barton, 1991; Barton *et al.*, 1974, 1992), has been successfully used in a number of relatively small diameter tunnels within the south-west region of the UK to assess rock mass quality and recommend subsequent stabilisation requirements. The tunnels were associated with the 'Clean Sweep' operation to improve the sewage/sewerage distribution within the south-west region (Clarke, 1993, 1997; Wetherelt and Eyre, 1997). Using data from the Gwithian outfall tunnel, this paper describes successful negotiation of 'extremely poor' and 'exceptionally poor' ground, provides back-analysis of a 'fall of ground' incident and comments on the use of the Q-system during tunnelling operations. Geological and geotechnical mapping of the face and roof of the tunnel was undertaken on a daily basis to assess rock quality and inform decisions regarding implementation of appropriate support. During tunnel construction, however, modifications to support categories were necessary to take into consideration ground conditions associated with major discontinuities and potential deterioration of mudstones. In view of the close proximity to the seabed, and the risk of water ingress, systematic probe (or cover) drilling was performed at regular intervals during tunnel construction. Where necessary, in order to reduce the water-make to within pre-defined limits, cementitious grouting of the ground ahead of the advancing face was undertaken.

2. Geology of the area

The tunnel was driven through the Gramscatho Group (Devonian) in a transitional zone between the sandstone-dominated Porthtowan Formation and the mudstone-dominated Mylor Slate Formation (Shail, 1989). Two major fracturing events have been identified in the immediate region (Alexander and Shail, 1995). Late Carboniferous to Late Permian extension created north-east–south-west (NE–SW) trending fractures and subsequent late Permian–Triassic extension that produced NW–SE trending fractures. Discontinuities identified in adjacent coastal exposures comprise

Figure 1. Map of Cornwall, showing location of Gwithian

bedding planes, primary and secondary cleavage, faults and joints. Faults, bedding and joints forming the rock mass fracture network collectively exert the most significant control on rock mass stability and permeability.

Previous mapping of adjacent coastal exposures identified several joint sets (striking NW–SE, NE–SW, E–W and N–S) in addition to bedding. NW–SE and N–S features tend to be tight and possess no infill, but where present it is likely to be quartz. In contrast, NE–SW and E–W faults exhibit greater aperture, up to 1 m in some cases, and commonly possess infill of quartz or breccia. Some discontinuities, notably faults, have high persistence, well over 100 m. Figure 2, using Dips software (Rocscience, 2013), shows an example pole contour plot of discontinuities identified during mapping of the tunnel at chainage 1700 m. Also shown on Figure 2 is the bearing or orientation of the tunnel (302°). Within the tunnel, bedding generally dipped at a low angle eastwards (10–20°). Variations in orientation of bedding were associated with either folding or close proximity to major discontinuities.

Five site investigation boreholes were drilled approximately 10 m away from the proposed route of the tunnel (including the shaft and final diffuser location). Uniaxial compressive strength testing of the core indicated moderate to strong rock at the tunnel horizon (mudstone and sandstone having strengths of 35 MPa and 90 MPa respectively). Estimates of rock quality from site investigation borehole data gave Q-values ranging from 'extremely poor' (0·028) to 'very poor' (0·33, 0·46 and 0·51) and 'poor' (3·0).

3. Application of the Q-system during tunnel drivage

3.1 Initial mapping and familiarisation

During early stages of the project, mapping of the shaft excavation and tunnel insert was used to gain an insight into rock quality and increase familiarity with the Q-system (Yelland, 1994). Initial estimates of Q were established together with the development of a standard recording procedure and mapping sheet or log. Table 1 provides a summary of the Q-system parameters (Barton *et al.*, 1974). Some difficulties were initially experienced with allocation of Q-parameter values: rock quality designation (RQD) was particularly challenging, due to cleavage. Orthogonal scan lines on the sidewall and face, combined with observations of the roof of the tunnel, were used to provide representative RQD values.

Owing to the nature of the construction work during tunnel drivage, access to the face was restricted, so it was imperative to develop a quick visual assessment of ground conditions. Face mapping was normally carried out on a daily basis, although actual mapping frequency depended on the number of drill and blast cycles completed. Face mapping included an assessment of the individual Q-parameters and a sketch of tunnel roof geology. Emphasis was placed on identification of key features likely to influence stability (for determination of joint roughness number, J_r, and joint alteration number, J_a). Additional factors considered during face mapping were

- potential variation of rock quality in tunnel side and roof, and allocation of representative Q-value for the mapping location
- influence of cleavage on RQD
- identification of likely failure mode (i.e. block fallout, slide etc., for assigning J_r and J_a)
- difficulties in defining number of influential joint sets, given limited tunnel exposure
- potential significance of major discontinuities on stress reduction factor (SRF), J_r and J_a.

3.2 Support categories and associated Q-values

From initial assessment of rock quality, carried out during the site investigation and surface mapping stage, support categories were established for the tunnel based on the Q-system, using a combination of bolts and steel-fibre-reinforced shotcrete (Sfr). During tunnelling operations, however, modifications to the support categories were considered necessary owing to exceptionally and extremely poor ground associated with major discontinuities and in areas of mudstone due to their potential deterioration under wet conditions. The final profile of the blasted tunnel was controlled by bedding and joint orientation (influenced by respective spacing and

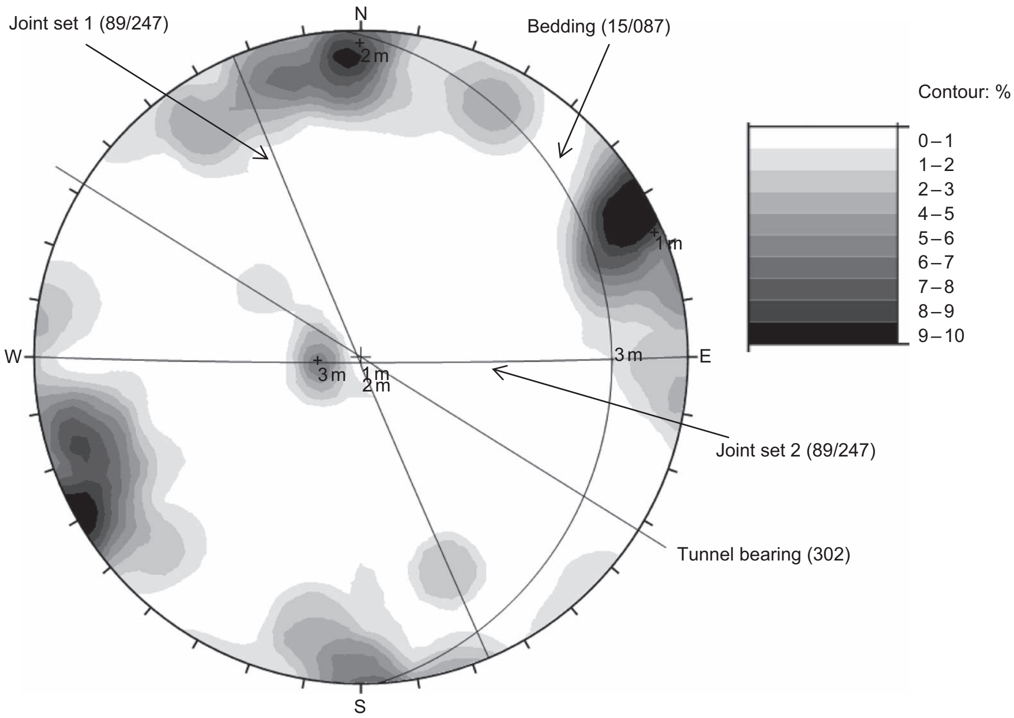

Figure 2. Pole contour plot of scan-line survey data from Gwithian outfall tunnel at chainage 1700 m, showing orientation of bedding and two joint sets and the bearing of the tunnel (angles given in degrees)

persistence, which control size and formation of adversely oriented blocks). Table 2 provides a summary of the support categories used during the tunnelling operation. Experience gained during tunnelling indicated that roofbolt installation was problematic in particular sections of the tunnel (Yelland, 1994). This led to the development of category 2A, incorporating fibre-reinforced shotcrete only. In 'exceptionally poor' ground conditions category 4A was used, with a lattice arch framework and a final 150 mm thickness of fibre-reinforced shotcrete. This replaced the original category 4, which was based on bolts. In areas dominated by mudstone, slight modification to category 1A was made to reduce the potential for ravelling of the upper sidewall by including shotcrete down the sidewall to axis level or below, as required.

Figure 3 shows the corresponding mapped length of tunnel associated with a particular Q-value. This indicates that the majority of the tunnel (95%) was driven in rock ranging from 'very poor' to 'fair' (Q-values ranging from 0·1 to 10 respectively). The typical range of mapped individual Q-parameters is provided below, for both general conditions encountered and typical fault zones.

Typical range of Q-parameter values

$$\frac{60-80}{9-12}\times\frac{1-1{\cdot}5}{2-4}\times\frac{0{\cdot}8-1}{2{\cdot}5-5}$$

Typical fault zone Q-parameter values

$$\frac{10-20}{15-20}\times\frac{1}{4-8}\times\frac{0{\cdot}6-0{\cdot}8}{5-10}$$

Evaluation of geotechnical data from the undersea tunnel suggests that, in general, the Q-system provided a sound basis for assessment of rock quality and for guidance on associated support requirements. However, sound engineering judgement informed the final support selection, particularly where 'exceptionally' and 'extremely' poor ground conditions associated with fault zones were encountered. Experience suggests that relying solely on Q-values may lead to potential problems, particularly in close proximity to major discontinuities. The Q-system does, however, by quantifying rock quality, provide vital information during the assessment process. It ensures that engineers take note of, and consider, factors that may affect

Q-parameter	Description
RQD	Rock quality designation
J_n	Joint set number
J_r	Joint roughness number
J_a	Joint alteration number
J_w	Joint water reduction factor
SRF	Stress reduction factor

$$Q = \frac{\mathrm{RQD}}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{\mathrm{SRF}}$$

Table 1. Q-system parameters (adapted from Barton *et al.* (1974))

rock quality and therefore control potential instability of the tunnel. Grimstad and Barton (1993) provide additional guidelines for support of narrow weakness zones using a 'practical mean Q-value' for the zone and the adjacent rock mass. Successful use of the Q-system clearly relies on the skill/experience of the geologist/geotechnical engineer undertaking the assessment.

In order to minimise blast damage it is important to undertake effective blast design. Generally, 2·7 m rounds were drilled during excavation of the tunnel. Smaller rounds were drilled in poor ground conditions. Ground conditions also dictated the number of holes drilled per round. In competent ground approximately 35 holes were drilled, whereas in poor ground as few as 16 holes were drilled. Correct timing of perimeter holes resulted in less damage to the surrounding rock. Figure 4

Q-value	Category	Support
$> 0{\cdot}8$	1	Spot bolt as required[a]
0·5–0·8	1A	Four bolts, 25 mm Sfr[b]
0·1–0·5	2	Five bolts, 50 mm Sfr
0·1–0·5	2A	50 mm Sfr (sides) 75 mm Sfr (roof) 100 mm concrete floor slab
0·01–0·1	3	Ten bolts, 25 mm Sfr
0·003–0·01	4	Ten bolts, 75 mm Sfr
0·003–0·01	4A	Steel arch, 1 m centres 150 mm Sfr 250 mm concrete floor slab
$< 0{\cdot}003$	5	Steel arch, 0·75 m centres 150 mm Sfr 250 mm concrete floor slab

[a]Bolts are 20 mm galvanised rebar, resin grouted, 1·8 m long, 900 mm spacing.
[b]Sfr is steel-fibre-reinforced shotcrete, dry mix application.

Table 2. Support categories and associated Q-values

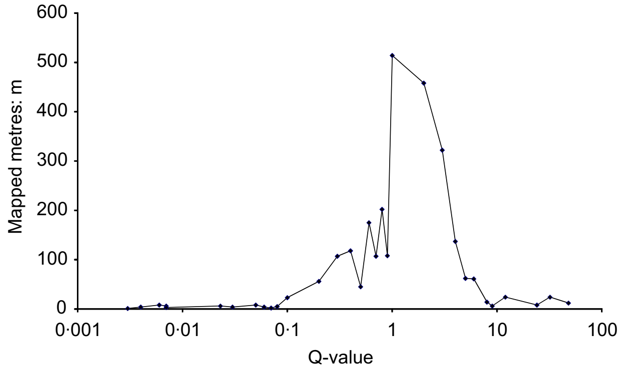

Figure 3. Distribution of mapped Q-values along the length of the tunnel

shows a reasonable correlation between weighted Q-values (taking into consideration the respective length of tunnel at a particular Q-value) and face advance rate per week. It should be noted, however, that face advance achieved is also influenced by potential delays resulting from installation of support and systematic probe drilling and grouting operations required for ground conditions ahead of the actual face position.

3.3 Probe drilling

During tunnelling operations a detailed log of key data was kept in the form of a roller graph, showing a plan of the tunnel which included: blast numbers, recommended support (Q), actual installed support, a brief geological description, probe face location and any grouting information.

From observations of local geology in coastal exposures and results from the series of packer tests conducted on the site investigation, borehole discontinuities would control the rock mass permeability. In view of the close proximity to the seabed, and the risk of water ingress, systematic probe drilling was performed at regular intervals during tunnel construction to assess ground conditions ahead of the advancing tunnel face. Where necessary, and in order to reduce the water-make to within pre-defined limits, cementitious grouting of the ground ahead of the face was undertaken.

Prior to probe drilling there was a need to establish a secure and competent face; in poor ground conditions this was undertaken with additional use of shotcrete. Under normal ground conditions probe drilling was undertaken every 25 m of tunnel advance. Where necessary, this distance or probe face frequency was reduced to reflect ground conditions. Standard procedure would be to drill the two side-horizontal holes numbered 4 and 8 shown on Figure 5. Holes 4 and 8, although drilled horizontally, were angled at 5° away (outwards) from

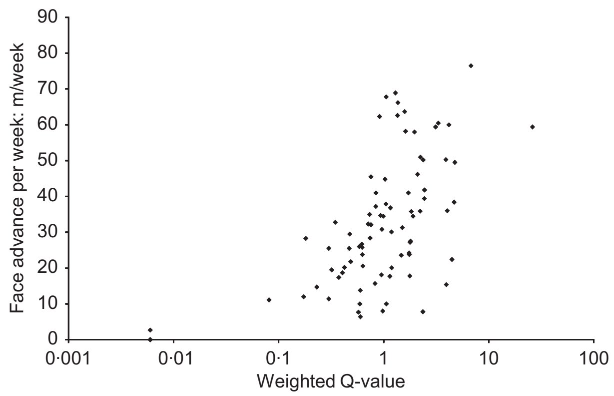

Figure 4. Effect of rock quality on weekly face advance rates

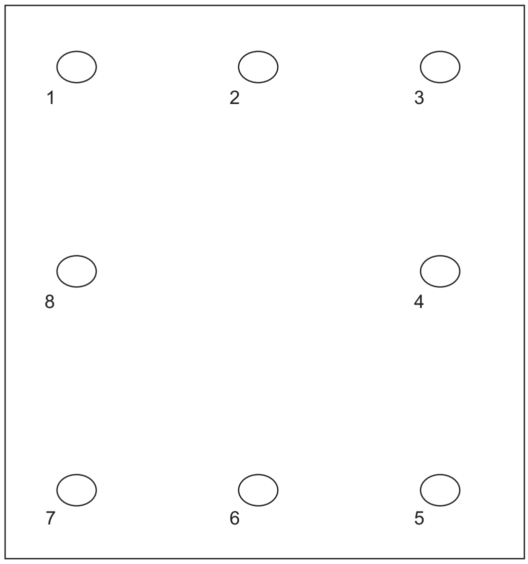

Figure 5. Simplified tunnel cross-section showing probe cover-drill pattern (not to scale)

the tunnel centreline. The length of the probe hole drilled was dependent on ground conditions and water-make, but restricted to a maximum of 39 m. Drilling rates and water inflow were recorded by the operator. Drilling rates gave an indication of likely ground conditions.

On completion of the two holes, packer-type standpipes were installed. Each valve was then opened and individual water-makes measured. If the collective water-make was less than 10 gallons/min (45 l/min), tunnel development continued as normal. However, if the collective water-make was greater than 10 gallons/min, the full face cover-drill pattern, shown in Figure 5, was then drilled. Top holes were inclined at 5° and bottom holes were drilled downwards at 5°. On completion of the eight holes, individual water-makes were totalled to provide a total collective water-make. Probe holes were then grouted by experienced operators with increasing pump pressures and grout thickness to seal the holes and reduce water-makes. On completion of grouting, two test holes were then drilled and the above procedure was repeated if the test holes made more than 10 gallons/min. Records were kept of the amounts of cement used for grouting. Secondary holes were drilled alongside original holes for the same distance and orientation. After drilling two sets of cover drilling patterns it was often necessary to establish a clean face should further drilling be required to reduce the overall water-make to an acceptable level.

4. Problems overcome during tunnel construction

4.1 Negotiation of 'extremely poor' ground conditions at chainage 2188 m

Figure 6 shows the variation in rock quality for a 300 m section of tunnel between chainage 2100 and 2400 m. This section was affected by two major fault zones: one at chainage 2188 m, which was associated with large quantities of water, and the other at chainage 2330 m, which was associated with 'exceptionally poor' ground conditions. By early identification of potential hazards these areas were successfully negotiated, although not without difficulty. The fault at chainage 2188 m required extensive grouting (approximately 60 t of grout carried out over various phases) from three probe face positions (2171 m, 2179 m and 2182 m) owing to high water-makes. Initial holes from the first probe face at chainage 2171 m were making in the order of 200 gallons/min (909 l/min), with collective water-makes in excess of 500 (2273 l/min) gallons/min (Yelland, 1994). Negotiation of the actual fault area (4 m thick, dipping at 45° to the south-east, containing ocherous clay and crushed earth-like material) was successfully achieved by reduced rounds (2·4 m, then 1·8 m drilled with a jack-leg machine), consolidation with shotcrete and installation of arches. Increased levels of support were also installed either side of the fault zone owing to the wet conditions. Category 3 support (based on bolting) was considered inappropriate for the fault zone because of the poorly consolidated ground, so category 4A was used. Ground conditions improved on the footwall side of the fault, but support modifications were also required on the sidewall on the excavation (where shotcrete was also placed down to floor elevation).

4.2 Negotiation of 'exceptionally poor' ground conditions at chainage 2330 m

A major fault, 5 m wide (containing chloritic clay infill), at chainage 2330 m, although relatively dry, also required

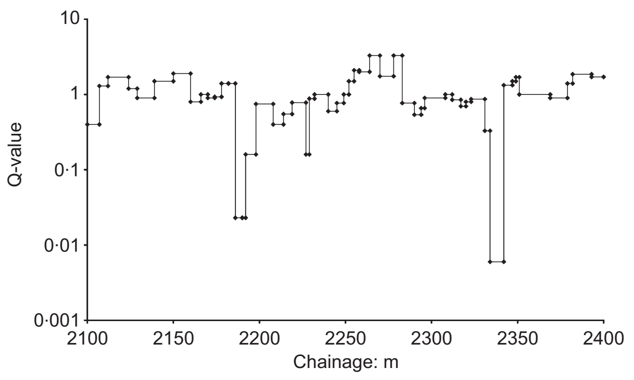

Figure 6. Variation of rock quality, Q-values, between chainage 2100 m and 2400 m

additional support measures because of the associated 'exceptionally poor' ground. Successful negotiation of the fault area is described below.

Probe drilling at chainage 2327 m indicated 'soft' ground conditions for 15 m followed by 'hard' conditions for the remaining 24 m. Both probe holes had minimal water-make (collectively around 1 gallon/min (0·4546 l/min)). Tunnel advance continued and the next round exposed fairly competent ground, but bedding plane dip increased to 40°. Increased support (greater than recommended by the mapped Q-value) was installed based on previous experience of fault negotiation. Further advance exposed exceptionally poor ground associated with the fault zone. Conditions deteriorated during installation of arch no. 7, which prompted the need for further shotcreting and building of the required floor slab. During stabilisation, however, further ravelling of the face occurred above arches 6 and 7, resulting in a 6 m void above the installed arches. The extent of the roof cavity was established from a series of probe holes drilled from chainage 2327 m, as depicted in Figure 7. A bulkhead was then established in front of arch no. 5 and the cavity was filled. Remedial support measures, including use of polyurethane foam for cavity filling, was required for ground stabilisation.

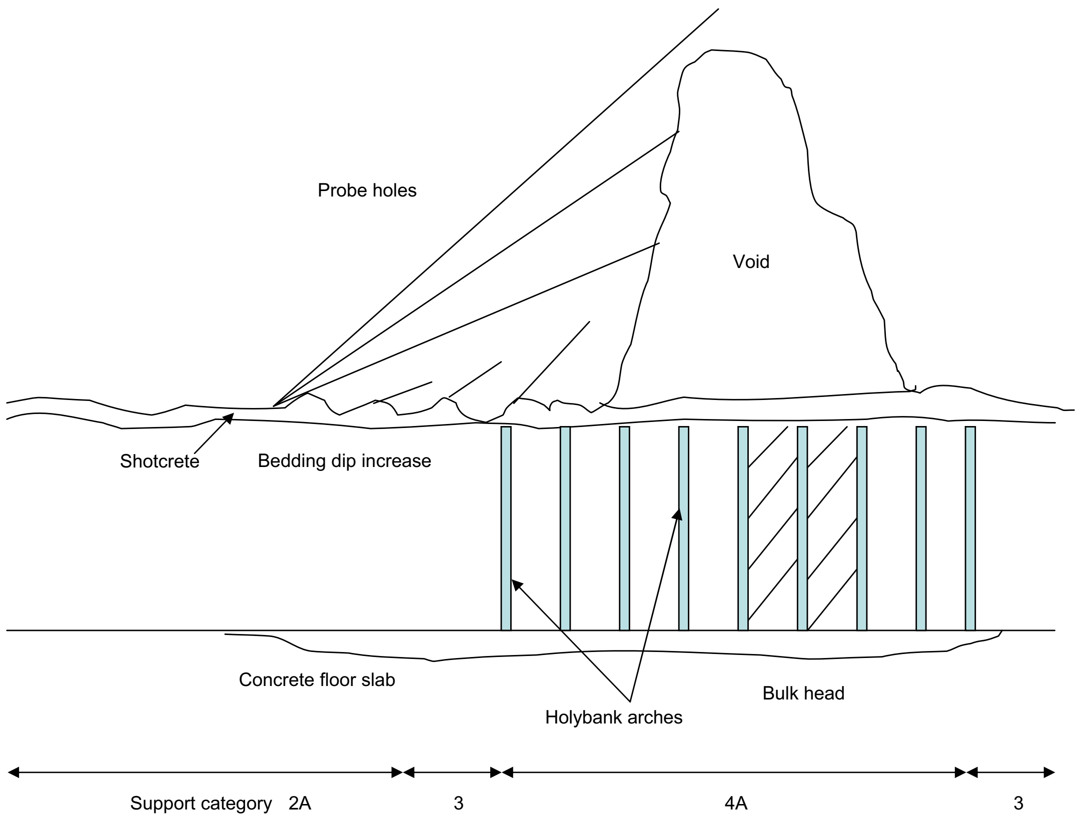

Figure 7. Longitudinal section of tunnel in vicinity of exceptionally poor ground around chainage 2330 m (not to scale)

Once the roof cavity had been filled, exploration holes were then drilled ahead of the face from the bulkhead to investigate ground conditions and determine the extent of the fault zone. Advance, through the bulkhead, required hand excavation, 'forepoling' and small box-headings in the upper right and upper left sections of the tunnel to expose the footwall of the fault. The upper right box-heading was completed prior to excavation of the second box-heading. On completion of the box-headings, arch crowns were then installed in the roof of the excavation and shotcreted into position. The remaining bottom bench was then removed in stages, with shotcreting undertaken prior to installation of spliced leg extensions to pick up the arch crowns. Following installation of a 200 mm floor slab, the whole area was shotcreted with a thickness of 150–200 mm. Ground conditions improved once the fault zone had been successfully negotiated.

This particular section of tunnel highlighted the need for additional support and modified excavation methods in the 'exceptionally poor' ground encountered. It should also be noted that, based on previous experience of successful fault negotiation and for precautionary reasons, a greater level of support was installed immediately prior to intersecting the fault than that recommended by the Q-system.

4.3 Back-analysis of a 'fall of ground' at chainage 1524 m

A fall of ground of approximately 2 t occurred at chainage 1524 m, resulting in damage to the cable reel canopy of the Tamrock drill-rig, and the vent duct was knocked to the floor. No persons were injured as a result of the fall of ground. A back-analysis of the incident was undertaken together with further mapping of the void created in the immediate roof. The face position was at approximately 1532 m when the fall occurred. The Tamrock drill-rig was drilling the face when the fall of ground occurred.

Initial mapping of the fall area during drivage of the tunnel had indicated the presence of coarse-grained sandstone and siltstone with bands of black shales between the bedding. Q-values had been fairly consistent throughout the area, ranging from 1·5 to 4·0, indicating 'poor' conditions, with support category 1 recommended (spot bolting where required). No support had been installed in the tunnel from chainage 1470 m.

Further investigation of the area after the fall identified that bedding orientation changed in the immediate location of the fall (from a dip and dip direction of 10°/300° to 20°/122°). Spacing of the bedding remained at 1·2–1·3 m. Importantly, 2–3 mm of clay infill was noticed on the regional jointing. 'Drippers' or minor inflow was also observed in the void created by the fall. A Q-value of 1·05 was determined for the fall area, and spot bolting (perpendicular to the failure plane)

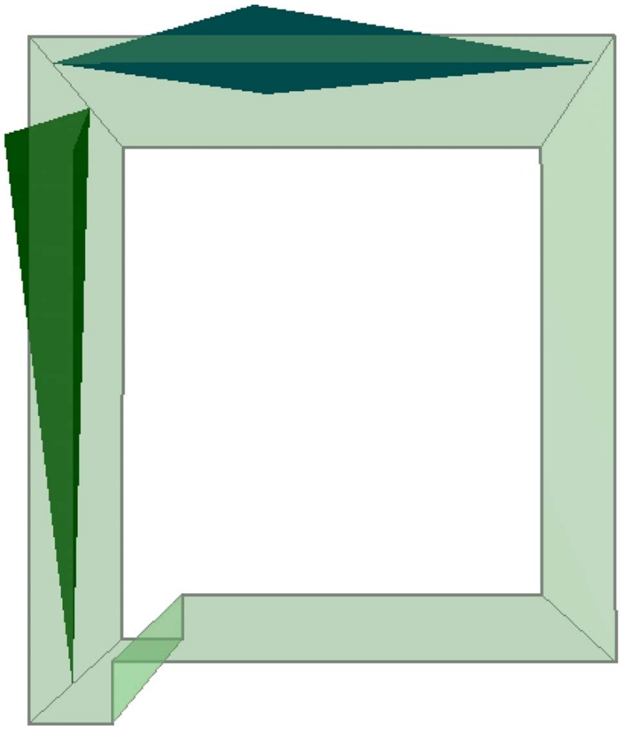

Figure 8. Potential wedges formed in roof and sidewall of tunnel from back-analysis of the 'fall of ground' at chainage 1524 m using Unwedge software

was subsequently installed from chainage 1522 m. Figure 8 shows back-analysis of the fall using Unwedge software (Rocscience, 2013), which confirms the potential for block fallout in the immediate roof and potential for wedge formation in the left-hand or southern sidewall of the excavation. The dip and dip direction of the discontinuities forming the fall of ground were: bedding 20°/130°, a clay-filled joint of 80°/070° and another joint of 90°/340°. Back-analysis of the incident suggests that stabilisation of the tunnel roof could have been achieved by either installation of strategically placed spot bolts or placement of an adequate thickness of shotcrete.

5. Conclusion

Successful completion of the Gwithian tunnel indicates that, when applied correctly, the Q-system provided a sound basis for assessment of rock quality and for guidance on associated support requirements. As expected, higher Q-values resulted in increased tunnel advance rates.

Experience indicates that relying solely on mapped Q-values may, however, lead to potential problems, particularly in close proximity to major discontinuities. Potential problems may also arise with interpretation of Q-parameter values in difficult ground conditions. This suggests there is still a need for training and education to ensure users are fully aware of limitations associated with application of the system. Initial familiarisation mapping under similar conditions was extremely useful during early stages of the project. It should be noted that rock mass classification systems do not identify potential failure modes or mechanisms of failure, so kinematic analysis of discontinuity orientation data is also required in low-stress environments.

The potentially subjective nature of rock mass classification data collection and associated interpretation highlights the need for suitably qualified and experienced site personnel for correct implementation. Importantly, engineering judgement informed final support recommendations.

REFERENCES

Alexander AC and Shail RK (1995) Late Variscan structures on the coast between Perranporth and St Ives, Cornwall. *Proceedings of the Ussher Society* **8**: 398–404.

Barton N (1991) Geotechnical design. *World Tunnelling*, November: pp. 410–416.

Barton N, Lien R and Lunde J (1974) Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics* **6(4)**: 189–236.

Barton N, Grimstad E, Aas G *et al*. (1992) Norwegian method of tunnelling, WT Focus on Norway. *World Tunnelling*, June: pp. 231–238.

Clarke I (1993) West Cornwall resewerage. *World Tunnelling*, December: pp. 449–453.

Clarke I (1997) Falmouth tank sewer. *World Tunnelling*, April: pp. 97–102.

Grimstad E and Barton N (1993) Updating the Q-system for NMT. In *Proceedings of the International Symposium on Sprayed Concrete – Modern Use of Wet Mix Sprayed Concrete for Underground Support*. Norwegian Concrete Association, Oslo, Norway, pp. 1–21.

Rocscience (2013) *Unwedge and DipS6.0 user manuals*. Rocscience Inc., Toronto, Ontario, Canada.

Shail RK (1989) Gramscatho–Mylor facies relationships: Hayle, South Cornwall. *Proceedings of the Ussher Society* **7**: 125–130.

Wetherelt A and Eyre JM (1997) Surveying for a ‘clean sweep’. *Proceedings of the 8th International Tunnelling Symposium, Tunnelling '97, Institution of Mining and Metallurgy, London, UK*, pp. 241–250.

Yelland RS (1994) *Case Example of Rock Mass Classification Applied to an Outfall Tunnel*. MSc thesis, Camborne School of Mines, University of Exeter, Exeter, UK.