Assessment of contour profile quality in D&B tunnelling

ASSESSMENT OF CONTOUR PROFILE QUALITY IN D&B TUNNELLING

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Abstract

Contour profile quality affects tunnel excavation costs, in terms of operational safety, support materials and construction time. In drill and blast tunnelling, under/over-excavation and rock mass damage arising from excavation phase can be evaluated by means of the elaboration of survey data and geophysical testing or coring, before and after the blast. As far as the quality of the profile is concerned, some indices can be used to define the contour and for the rock mass in the boundary as well.

This paper proposes a methodology well applicable to rock tunnelling, and a case study based analysis to correlate the over-excavation and the rock mass conditions is discussed to validate the procedure. Profiles and geological parameters have been processed with automatic code specifically developed for the study. Over-excavation distance and Tunnel Contour Quality Index are evaluated and compared with Q-system values. The results have been discussed, compared with other literature cases and validated for engineering applications.

Keywords: controlled blasting, contour evaluation, TCI, BDI, overbreak

1. Introduction

The quality of the excavated contour in underground tunnel directly affects final costs of the infrastructural facilities (*Scoble et al.*, 1997*; Hu et al.*, 2014). Poor contouring can produce under or over-excavation and artificial fractures into the rock mass. These factors produce many unfavourable consequences: scaling or specific supports are required, advancing rate decreases, convergences may increase, time schedule increases and safety is compromised. Directly related to the convergences and safety, also static approval tests are facilitated by a good contour profiling: in fact, both first phase lining and final lining are affected in terms of thickness, strength and durability (*Pelizza et al.*, 2000).

Rock mass conditions are an essential factor in choosing the adequate excavation method (*Mahdevari et al*., 2013); drill and blast (D&B) technique is the most appropriate in rock masses that present high compressive strength and that are abrasive (*Cardu et al*., 2004). Contour quality in D&B tunnelling depends on many factors: geological properties and conditions (e.g. rock mass quality and stress), blast design and drilling pattern execution (*Oggeri and Ova*, 2004; *Singh and Xavier*, 2004; *Singh et al*., 2013). Initial rock mass conditions depend on the site geology, but drilling operations and blasting round affect the rock mass structure because of vibrations, shock wave propagation, gas pressure and stress redistribution (*Singh et al*., 2003; *Hu et al*., 2014). These factors act on the rock mass depending on the microstructural fabric orientation (*Nasseri et al.*, 2011) and pre-existing fractures. Charge per delay and total charge per round must be adequately set to preserve rock mass integrity or avoid previous fractures worsening. Charge limit criteria cannot be based on the peak particle velocity (PPV) values as it happens for the man-made structures, because the limit charge is usually determined to control excessive vibration consequences at distance (*Cardu et al.*, 2004). However, even if approximated from elastic media and pure compression waves, PPV relates the acoustic impedance with the stress level that the blast produces because of rock type, stress conditions, rock properties (i.e. density, porosity, anisotropy), water content and temperature (*Singh et al.*, 2003). Blast sequence directly affects the extension of induced fractures; all blasting (contour, production, smooth) in each round produce a cumulative damage effect, both with smooth blasting or pre-splitting method. However, the two methods present some differences in the orientation and intensity of the damage that they generate. The smooth blasting produces both columnar shaped elements finely spaced and also widespread micro cracks; while in the pre-splitting the formation of columnar steep elements is more extended (*Hu et al.*, 2014).

Taking into account the importance of the determination of rock damage and contour conditions after a D&B tunnelling, related to rock mass geology, geostructural features, drilling pattern and blasting sequence, this paper focuses on the assessment on the quality of the tunnel profile by means of some indices. This can be done using quick, easy to find and reliable profile survey techniques, properly adjusted and whose data can be processed to let a practical tool available for technical control and also to limit contractual disputes.

2. Damage indices

Damage in rock mass means a drop of strength, caused by the opening or shearing of new or extended cracks and joints (*Scoble et al.*, 1997). It can affect both underground and open pit excavations and it is related to the previous discontinuities conditions. The blast produces a direct damage around the blastholes and also an indirect damage due to vibration and rock block dislocation. Vibrations and explosive detonation products can propagate fractures into the rock mass and open existing joint, and this can induce an excavation disturbed zone (EDZ). This zone is the resulting volume around the tunnel boundary, whose extension depends on the excavation method, also valid for the extent of non- blasting methods (*Barton*, 2007), affected by damages due to excavation and disturbance due to stress state modification. Considering underground tunnelling, the damage can be generally divided in three classes:

- Major damage: when there is rock falling from tunnel roof and/or pillar.
- Minor damage: when there is chips detachment from tunnel roof and/or pillar.
- No damage: when there is not visual damage.

Various techniques can be used for the rock damage evaluation, some were developed for particular studies, and others are used during excavation routine (*Scoble et al.*, 1997; *Singh and Xavier*, 2005):

- Assessing pre-blast: the inherent damage is evaluated, constructing a geomechanical classification (i.e. Bieniawski's classification) in order to build a base reference for post blast.
- Visual inspection and survey: provide qualitative information on pre/post blast damage and a rock mass classification. Also a borehole camera can be used for core assessment.
- Traditional observation methods: give an indirect measurement of damage. Usually the Half-Cast Factor (HCF) or scaling time is used.
- Rock mass classification methods: empirical rock mass quality rating systems (e.g. Q-system), inherent-damage index and blast-induced damage (e.g. Blast Damage Factor, Blasting Damage Index).
- Geophysical methods: such as seismic tomography, loose rock detection sensors and ground-penetrating radar, high-frequency cross-hole seismic, seismic-refraction tomography.
- Vibration analysis: the damage in the near-field is evaluated from peak particle velocity (PPV) values and rock mass strength.

Four main indices are available for this evaluation: Blast Damage Factor, Blast Damage Index, Failure Approach Index and Tunnel Quality Index, that are briefly illustrated in the following sections. They do not describe the geometrical condition of the excavated contour, which depends on the comparison with the design profile, but they focus on the rock mass damage. During an underground excavation, each blast round is individually mapped, in order to evaluate or update the required support (*Barton et al.*, 1995) and to modify, if necessary, drilling pattern and blast design.

The Q index has been the one used in this study because of the available data. Anyway, the others are presented here in order to provide a more complete overview on the available indices. These could be used in further work if the data collection will take their parameters into account.

2.1 Blast Damage Factor

The Blast Damage Factor *D* (*Hoek et al.*, 2002; *Hoek*, 2012) is a parameter introduced in 2002 into the Hoek-Brown failure criterion. It estimates the global rock mass strength and the rock mass modulus. Its range falls between 0 (undisturbed rock mass) and 1 (highly disturbed rock mass). This parameter must be set only for the actual zone of damage, not for the entire rock mass surrounding the excavation and the definition of this extension represents a meaningful assessment. Ideally, the volume between front and undisturbed rock mass can be divided into a number of layer with different values of *D* using numerical modelling, but usually a single *D*-value is set for practical reasons. The production blasting data help to determine the actual damaged volume; some outlines (*Hoek et al.*, 2002) suggest the right *D*-value by giving a description of the rock mass and its appearance. Figure 1 – 4 show some examples for D&B tunnelling (and also one example of mechanized underground excavation).

Figure 1. Primolano tunnel (Italy). High quality of the tunnel contour, half blasthole clearly evident at ribs and crown. Suggested *D* = 0. (Courtesy Italesplosivi)

Figure 2. Irregular tunnel contour after D&B; shotcrete for the first phase support is smoothing asperities and over excavation, but nominal profile is not obtained yet. Suggested *D* = 0.7. (Anonymous)

Figure 3. Hydropower tunnel in Northern Italy. Local damage and irregular profile at rib is due to spalling in metamorphic rock mass and anisotropic state of stress, even if mechanised tunnelling with a full face open TBM has been adopted. Suggested *D* = 0.7. (Photo by C. Oggeri)

Figure 4. Very irregular profile after D&B tunnelling due to rock joint pattern and poor contouring techniques. Suggested *D* = 0.8. (Photo by C. Oggeri)

2.2 Blast Damage Index

Blast Damage Index (*BDI* - Equation 1) was developed by *Yu and Vongpaisal* (1996) for mining works. This relation takes into account the mechanics and the effects of wave propagation into the rock mass: the compression wave arrives at the free surface and is reflected as a tensile stress wave that causes the damage (*Barton*, 2007). They analysed how much mining work affects slope walls and roof stability. *Cardu et al.* (2004) used this index to assess rock slope induced damage along mountainsides, when the advancing face of a tunnel approaches the external slope.

$$BDI = \frac{IS}{DR} = \frac{VdC}{KT} \tag{1}$$

Where:

- *IS*: induced stress.
- *DR*: damage resistance.
- *V*: vector sum of PPV (mm/s).
- *d*: specific gravity for rock mass (kg/m^3).
- *C*: compression wave velocity of rock mass (mm/s).
- *K*: site quality constant.
- *T*: dynamic tensile strength of rock mass (N/m^2).

Yu and Vongpaisal (1996) and *Singh et al.* (2003) assume the value of *RMR* (Rock Mass Rating) as site quality constant *K*, that is the most adopted.

Table 1 shows a comparison between *BDI* ranges. Mining works presents higher limits of BDI than mountainside places (*Cardu et al.*, 2004) but in both cases varies from zero to one. *Singh et al.* (2003) recommended very different limits in a coal mine situation.

Table 1. Comparison of *BDI*s in mining works *Yu and Vongpaisal* (1996), mountainside cases (*Cardu et al.* 2004) and coal mine *Singh et al.* (2003).

BDI	Mining	Mountainside	Coal Mine
Absolutely safe	< 0.125	< 0.060	< 1
No noticeable damage/falls seldom	< 0.250	< 0.200	< 2
Serious problems	> 0.250	> 0.200	> 2

2.3 Failure Approach Index

Failure Approach Index (FAI) proposes a quantification of the rock mass damage when numerical simulations are used in tunnel support design (*Xu et al.*, 2017). At the beginning it was developed for plastic behaviour but then it was improved (*Xu et al.*, 2017) for an elastic-plastic model that takes into account the relation between stress and strain, the strength criterion and also the post-failure response, considering isotropic conditions. This index is proposed for interlayered rock (FAI_m) and bedding plane (FAI_j) in order to find the layered rock mass FAI, which is the maximum between those two.

2.4 Q-system

Tunnel Quality Index (Q-system) is a consolidated and suitable rock mass classification system developed by *Barton* (1974). It is extensively used in underground rock engineering application and it allows also some correlations to empirically estimation of rock mass properties.

There are some parameters that need careful evaluation in order to improve the accuracy of Q-system; among the others, joint orientation is related to tunnel axis orientation but it is not numerically ranked. In fact the numerical evaluation of this parameter would make the classification less general. Moreover, joints and their characteristics are often difficult to be correctly determined: they form complicated three-dimensional patterns in the crust, while surveys are made in surfaces (two-dimensional) or boreholes (one-dimensional) (*Palmstrom*, 2005).

3. Overbreak evaluation

Tunnel excavation quality depends also on the contour geometry. Overbreak or bad profiling directly affects construction costs: more supports are required to avoid that some rock falls and more concrete is necessary to fill up empty spaces in order to help covering layer installation (*Scoble et al.*, 1997). Furthermore, the type and quantity of supports (preliminary and long term layer) affects static approval tests, both during construction stage and long term monitoring (*Pelizza et al.*, 1999; *Pelizza et al.*, 2000)

Some key indicators can be used:

1. Overbreak area (*Mahtab et al.*, 1997*; Mandal and Singh*, 2009). It is the excavated section area that exceeds the design (or paid) tunnel section. It is evaluated on a percentage on the design section area.
2. Overbreak distances (*Kim*, 2009*; Olsson*, 2010). It is the distance between design and excavated contour.
3. Tunnel Contour Quality Index (*Kim,* 2009*; Kim and Bruland, 2010; Kim and Bruland,* 2015). This index relates overbreak distances, contours ratio and longitudinal variation in each blasted round. It can also be evaluated for the entire tunnel.

3.1 Overbreak area indicator

The magnitude of overbreak can be defined as the difference between design and excavated sections. It allows to evaluate the volume of rock that exceeds the planned mucking. In order to consider comparable data, the overbreak area ($O_{v\ area}$) is evaluated in percentage (*Mahtab et al.*, 1997; *Mandal and Singh,* 2009) as the difference between excavated (A_e) and design (A_d) tunnel section, normalized on the design section area (Equation 2):

$$O_{v_{area}} = \frac{A_e - A_d}{A_d} \times 100 \qquad (2)$$

Mandal and Singh (2009) also propose to divide the cross-section in three zones, in order to evaluate the impact of different stress path and blast design effects. This approach demonstrates that the crown is more affected by overbreak, due to its stress conditions, claiming for a particular attention on the drill plan and the blast design of this zone.

3.2 Overbreak distances and damage distances indicator

In construction manual guidelines and contractual claim, the overbreak is generally evaluated as the distance between design and excavated contour (*Mahtab et al.*, 1997; *Scoble et al.*, 1997; *Kim and Moon*, 2013; *Konkan Railway Corporation, 2012*). This approach allows to elaborate directly the topographic mapping data, which is more intuitive than the overbreak area approach. The admitted overbreak distance depends on the position of the section in the blasting round. In fact, the drilling look out angle makes the excavated contour bigger at the end of the round than at the beginning. The admissible overbreak distance can be evaluated as a mean of the distance at round beginning and at round end (*Olsson*, 2010 - Figure 5).

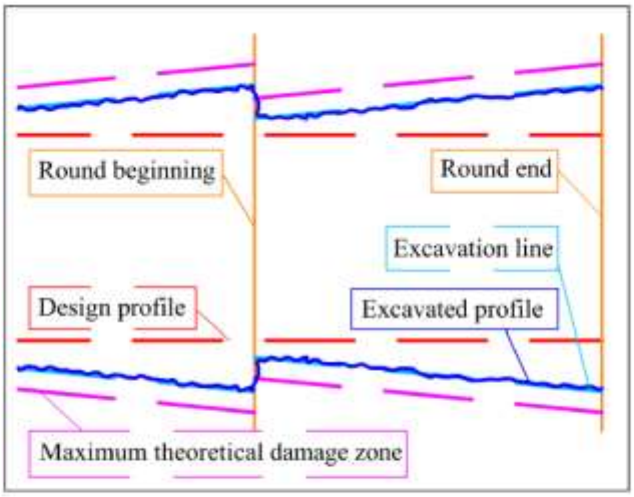

Figure 5. Scheme of plan view. Excavated contour compared with design contour all along one round (modified from: *Olsson*, 2010). The start cross section is smaller than the end one due to drilling lookout that is necessary to have enough operative space.

The maximum overbreak distance (O_v) depends on each national legislation and special conditions can be arranged between the parts in the contract (*Olsson*, 2010; *Konkan Railway Corporation*, 2012). Scandinavian countries present similar values of the admissible overbreak distances. Table 2 shows a comparison of the excavation classes used in Sweden (Anlaggnings AMA) and Finland (InfraRyl) (*Olsson*, 2010).

Table 2 Excavation classes of tolerance in Sweden and Finland (from: *Olsson, 2010*).

Excavation tolerance	Maximum admissible distance expressed as

classes	average of round beginning and round end [m]		
	AMA - Sweden	InfraRyl - Finland	
		Walls and crown	Floor
1 - special class	0.30	< 0.20	< 0.20
2 - normal class	0.35	< 0.40	< 0.60
3 - tunnel access (first 10 m)	0.40	< 0.60	-
4 – special cases		No demands	No demands

Norwegian and Italian legislations come from the Swiss one (*SN*, 2004; *NPRA*, 2012). In these countries the overbreak (O_v) depends on the theoretical excavated area (A_d) using Equation 3 as shown in Figure 66:

$$O_v = 0.07 \times \sqrt{A_d} \qquad (3)$$

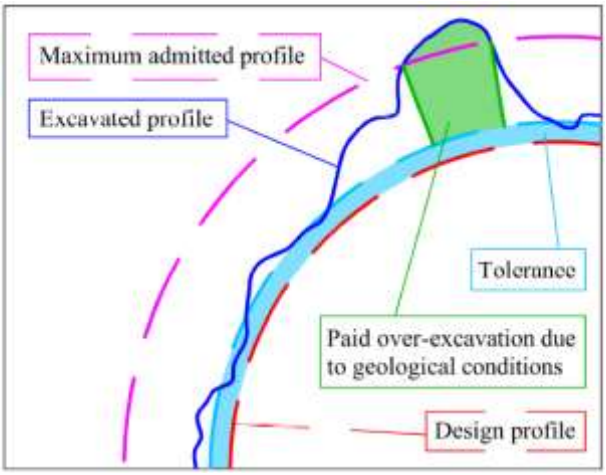

Figure 6. Contractual profiles: the design profile presents a tolerance (light blue area) but the admissible over-excavation is measured from the design profile (not from the tolerance) and depends on the minimum between Equation 3 and 0.4 m. Outside the maximum admissible contour the over-excavation costs relapse on the contractor but the over-excavation costs due to geological condition relapses on the client (green area) (modified from *SN*, 2004).

All the evaluations on overbreak distance consider it outside the design contour because no rock within the design profile is admissible (*Mahtab et al.*, 1997*; Olsson*, 2010).

3.3 Tunnel Contour Quality Index

This index was developed (*Kim*, 2009; *Kim and Bruland*, 2010; *Kim and Bruland*, 2015) in order to evaluate tunnel and rounds contour quality in D&B context. This index takes into account overbreak distances of each cross-section (O_v), contour roughness as ratio of contour length (*RCL*) and longitudinal overbreak variation (V_0), as shown in Figure .

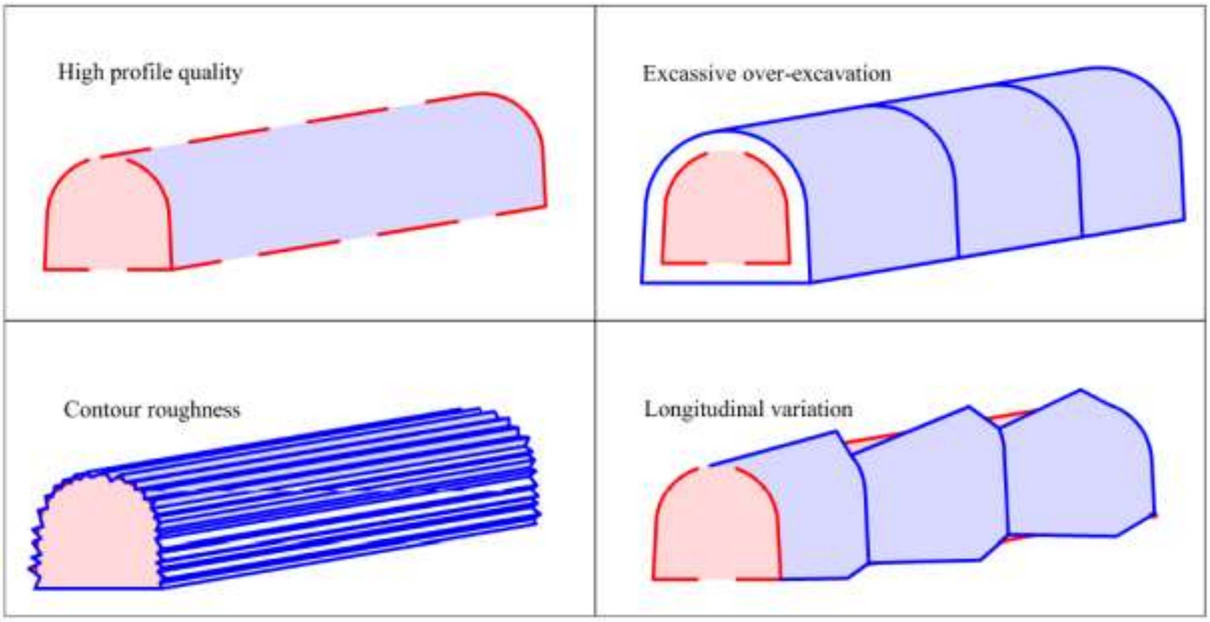

Figure 7. Different effects of the parameters that affect the TCI (modified from: *Kim*, 2009). Excessive overbreak directly affects muck volumes and final lining to reach the design contour; contour roughness influences lining and supports and can cause under-excavation; longitudinal variation affects the operations of lining placement.

Equation 4a relates these parameters for the entire tunnel where more than five consecutive rounds are available, Equation 4b can be applied in each single round.

$$TCI_{tunnel} = \frac{C_r}{W_1 C_1 \overline{O_v} + W_2 C_2 \overline{RCL} + W_3 C_3 V_0} \quad (4a)$$

$$TCI_{round} = \frac{C_r}{W_1 C_1 O_{v\_round} + W_2 C_2 RCL_{round}} \quad (4b)$$

Where:

- C_r: constant of adjustment.
- W_1: importance of additional mucking.
- C_1: overbreak correction factor.
- $\overline{O_v}$: average of the rounds overbreak distance [cm].
- W_2: importance of additional shotcrete.
- C_2: contour lengh correction factor.
- $\overline{RCL}$: average of the round contour ratio.
- W_3: importance of longitudinal variation.
- C_3: longitudinal overbreak correction factor.
- V_0: longitudinal overbreak variation [cm], which is the round overbreak standard deviation.

The total overbreak is calculated with the following steps:

1. O_v: distances between excavated contour and design contour in many points of the same cross-section.
2. $O_{v\_section}$: average value of overbreak distances (in cm) of each scanned section.
3. O_{v_round}: average value of $O_{v\_section}$ considering at least two sections in each round.
4. $\overline{O_v}$: average value of O_{v_round} to consider the entire tunnel.

Figure 8 shows the practical way of the procedure.

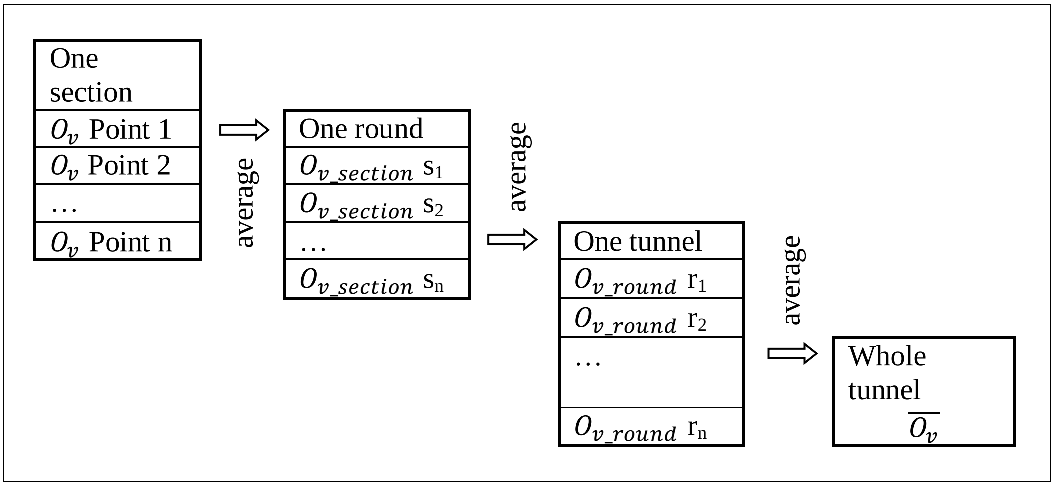

Figure 8. Scheme of the procedure that is necessary to calculate average overbreak, one of the main TCI parameters. The procedure to obtain rounds and tunnel RCL is almost the same.

Coefficients of adjustment have been calculated following the procedure well explained in *Kim* (2009). Equation 5 gives an example of their structure showing the overbreak coefficient equation.

$$C_1 = 1 \Big/ \left[\left(\frac{1}{n} \sum_1^n O_{v\_round} \right) + 5 \times std(O_{v\_round}) \right] \tag{5}$$

3.4 Profile survey

After rounds blasting, the geometry documentation is important both for owner and contractor, in order to evaluate excavation quality, excavated volumes and supports (*Gikas*, 2012). Contact (finger probes, tape extensometer and section profiler) and non-contact instruments (theodolite, total stations, photogrammetry, optical triangulation and Terrestrial Laser Scanning – TLS) permit data acquisition (*Pejić*, 2013).

The methods that are mainly used are photogrammetric techniques, Terrestrial Laser Scanner (TLS) or conventional survey with total station (*Olsson*, 2010; *Gikas*, 2012).

The photogrammetric techniques can give a 3D scan of the tunnel tube collecting each surface point at least in two photographs. It is a quite low cost technique but is not common in underground works because the surface is irregular and there is not enough light for taking quality pictures .

The Terrestrial Laser Scanner (TLS) can rapidly locate points with high accuracy (e.g. thirty meters can be scanned in ten minutes) and it provides a point cloud. The presence of reflective objects (e.g. equipment and water) can affect the recognition of targets (*Gikas*, 2012). Data can be shown in a virtual reality model and, if texture information are available, it is possible to render a photorealistic VR model (*Chmelina*, 2010).

The total station needs a calibration and some starting parameters are set manually: profiles interval, measuring angle, beginning and ending chainages; then total station could reveal points automatically with iterations. The instrument should be located as near as possible to the symmetry axis, in order to equilibrate the density of the points on the contour; this surveying method took about one hour each ten meters of tunnel. This procedure is usually done after scaling and shotcreting for safety reasons. Also the total station survey can be affected by the presence of reflective objects (e.g. equipment and water) as the TLS can be.

A profile-image method can also be used in tunnel works (*Wang et al.*, 2009; *Wang et al.*, 2010), joining laser profiling with photogrammetry to obtain a more accurate survey.

4. Methods

4.1 Data gathering

The study case is a roadway tunnel excavated in one of the North provinces of Norway. The tunnel is 4585 m long with a face of about 80 m^2 of surface and lies under 300 m of overburden. The work was planned for a period of nine months (from August 2014 until April 2015). The tunnel was excavated by D&B using the Norwegian Tunnel Method of Tunnelling (*NTNU*, 1995) - NTM - that is the application of New Austrian Tunnelling Method - NATM- on hard rock. The construction was developed through competent metamorphic rock mass, composed by sandstones, slates and expansive clays with chlorites. The Q index, obtained from visual inspection of the tunnel face, was used as classification of the ground condition. The available data from geological site survey list 54 rounds located from kilometric point KP 5561.9 to 5781.8 (47 surveys) for Portal 1 and from KP 10127.5 to 10107.5 (7 surveys) for Portal 2 of the tunnel. Three main joint set families were observed along the rounds excavated (as from the geotechnical report). Table 3 describes the relative range of dip and dip direction of these main joint sets with respect the tunnel axis.

Table 3. Basic data for the main joint sets.

Orientation of the main joint sets observed	
Joint set (S)	Dip / Dip Direction (°)
S1	45 – 70 / 235 - 255
S2	60 – 80 / 010 - 020
S3	40 – 65 / 095 - 120

Two jumbos Atlas Copco XE3C and XE3D of three booms each, equipped with percussive-rotary top hammer drilling mechanism, working in semi-automatic ABC total system were used to drill the analysed blast rounds. Available data concern production face drilling holes of short length (4.0 – 5.5 m), drilled by using only one rod (5.5 m length and 38 mm diameter) and a bit of 46 mm diameter.

The charging of the blastholes was carried out with emulsion of different lineal charge according to the type of blasthole; nominal charging information estimate theoretical lineal charges of 1.6 kg/m, 1.2 kg/m, 0.85 kg/m and 0.5 kg/m for cut/lifter, stopping, second contour and contour holes, respectively. The nominal number of blastholes per round was about 140; this counted about 16 cut holes, 12 lifter holes, 57 stopping holes, 24 second contour holes and 32 contour holes. Stemming was estimated in 0.4 m for all blastholes with exception of contour holes that were not stemmed. The firing was bottom initiated with booster and non-electric detonators; nominal timing reports indicate the use of LP non-electric detonators from numbers 0 to 60. Round progress was 93% of the drilled length and production was estimated at 1.6 blasts per day; this is a progress of about 7.2 meter per day. The excavation was made simultaneously from the two sides of the tunnel.

Portal 1 and 2 were located at 105 m a.s.l. and 6.75 m a.s.l. respectively. Starting from Portal 1, the tunnel was upwards oriented with a slope of 1.5% for about 617 m, followed by a downhill of a -2.5 % slope until Portal 2. Tunnel cross section dimensions were decided considering the estimation of traffic volume twenty years after the opening (Annual Average Daily Traffic volume – AADT) and the tunnel length (*NPRA*, 2004); AADT was estimated between 7500 and 9500 units. In order to fulfil this traffic volume, most of the cross-sections follow the Norwegian type section T9.5 (theoretical excavated area 74 m^2) apart a widening zone of the tunnel that follows the cross-section T12.5 zone (theoretical excavation area 100 m^2), for a length of about 30 meters. The face area of the two transition zones, before and after the widening, 30 m long each, was increased (or decreased) regularly until matching the respective cross-section, namely that of T9.5 and T12.5.

Table 4 shows the measurements for the used cross sections, referred to Figure 9. The design area starts to increase from chainage (KP) 5785 until KP 5815 to reach the T12.5 section between KP 5815 and KP 5845. Then it decreases until KP 5875.

Topographical mapping of the excavated void after blasting was surveyed with a total station Leica Viva on the shotcreted surface; set angle was chosen to reveal approximately one point each 50 cm on the contour. Typical thickness of the shotcrete liner lies between 80 and 100 mm.

Cross-section profiles perpendicular to the direction of the tunnel axis of the excavated face were extracted at every 1 m in AutoCAD files. Each profile was identified by its respective kilometric point; this comprises a total of about 500 excavated profiles measured.

Table 4. Geometric measurements for tunnel cross-section that was used in this case study (*from: NPRA*, 2004*; Tunnel project documents*).

Geometric measurement	Section T9.5	Section T12.5

Total width (B_T)	9.50 m	12.50 m
Carriage way width (B_K)	7.00 m	10.00 m
Lanes	2 × 3.50 m	3 × 3.50 m
Shoulder (in verge area)	2 × 1.00 m	2 × 1.00 m
Sidewalk (in verge area)	2 × 0.25 m	2 × 0.25 m
Centre point wall radius (X)	0.44 m	3.44 m
Centre height wall radius (Y_V)	1.57 m	1.57 m
Wall radius (R_V)	4.79 m	4.79 m
Centre height lining radius (Y_H)	1.22 m	-0.46 m
Lining radius (R_H)	5.20 m	7.45 m
Vertical clearance	4.60 m	4.60 m
Nominal area	66.53 m^2	91.23 m^2
Concrete upholstery	0.60 m	0.60 m
Excavation area	74m^2	100m^2

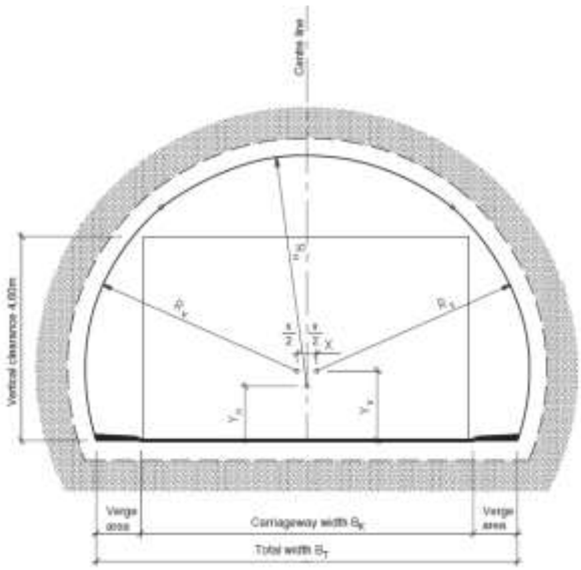

Figure 9. Geometry of tunnel cross-sectionT9.5 and T12.5 (*NPRA*, 2004).

4.2 Data analysis

The aim of the survey is to assess the quality of the resulting contour from the excavated profiles compared with the theoretical section intended. Since the actual lineal charge of the holes is not available, the explosive is considered as a constant variable. Therefore, differences in the excavation sections (over-excavation and under-excavation) should be mainly generated by a variation in the geotechnical rock characteristics. The work developed by *Costamagna* (2016) is applied here in order to evaluate round blast results in terms of overbreak and TCI. For this reason, a Matlab script has been developed in order to automatically make uniform and treat three kinds of data (Figure 10):

1) Scanned profiles (about 500 *.dxf* files)
2) Geotechnical characterization (54 surveys)
3) Drilling data (measurements while drilling, about 11700 *.MWD* files)

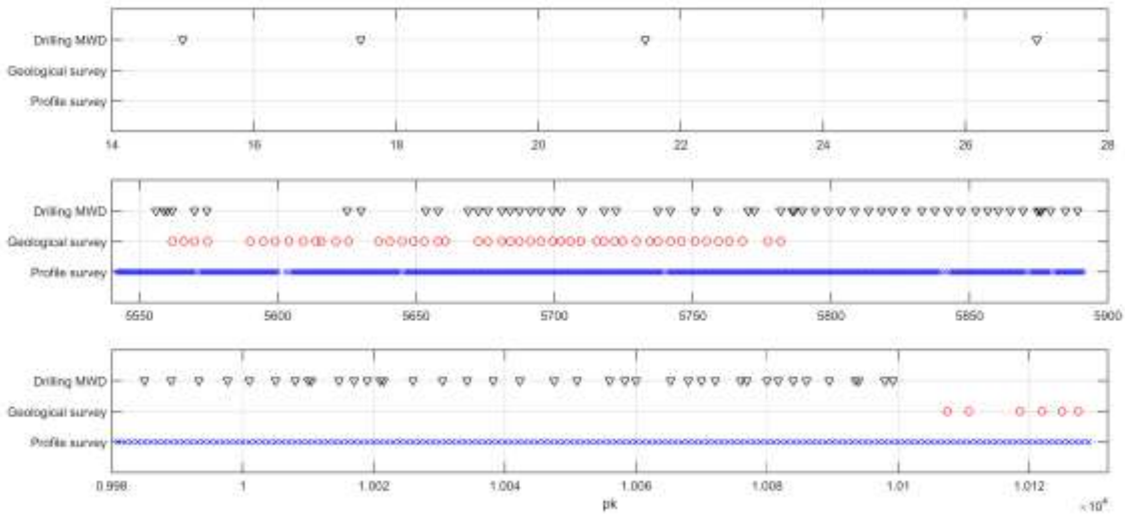

Figure 10. Analysed data as function of the chainage. The blue crosses show the scanned cross-sections, red circles show the beginning KP of the geological surveys and black triangles show the nominal position of start drilling sections where MWD data are collected.

The data collected by means of the geostructural survey allowed to set the Q-system parameters. The other rock mass damage indices (i.e. BDF, BDI, FAI) could not be evaluated in this case study.

In order to evaluate round results and compute some of the parameters used to assess TCI, it is necessary to identify each round and the cross-sections that belong to each of them. The KP of a new round is measured topographically. In the case study, the KP was measured both manually, by using a total station (used to reference the geotechnical reports), and automatically through the MWD system. The drilling jumbo has a laser scanner installed in its front side. During the positioning of the jumbo and before the drilling of a new round starts, the jumbo is aligned with the tunnel axis, by making pass through two targets located in one of the boom a laser beam aimed in the direction of the tunnel axis. The laser scanner also measures the distance to the face of the new round and records the kilometric point (here intended as the nominal KP) at which it is located inside the tunnel.

When MWD has been correctly measured and recorded, the mode of all *z* coordinates (borehole position along the longitudinal axis referenced to the nominal KP) in each drilled section is added to their nominal KP, in order to consider also the irregular face surfaces.

In case no MWD data is available for adjacent rounds, the KP taken from the geotechnical reports is used. KPs from the MWD system in which *z* coordinates records have failed for all boreholes and taken from the geotechnical reports may induce some error in the beginning of the round (due to irregularities in the face) and also occasional overlapping between two adjacent rounds (as for the 93 % round progress). Since cross-sections are scanned at every 1 m depth, a correction for clustering excavated areas between two adjacent rounds has been carried out by adding a length of 0.5 m to the initial KP, in order to reduce these KP errors. In addition, rounds shorter than three meters have been rejected because at least three cross-sections in each round are necessary for the round analysis and the maximum length per round has been identified by using a robust variance estimator (*Miller and Miller*, 2010) and considering site work conditions. Therefore, rounds between 3 to 5.5 m lengths have been considered for the analysis; this comprises 84 available rounds. Scanned cross-sections within the kilometric points of two adjacent rounds of the MWD system will be framed in their respective round. Finally, the first and the last profile of each round have been discarded because their blasting results can be affected by the above errors commented (Figure 11).

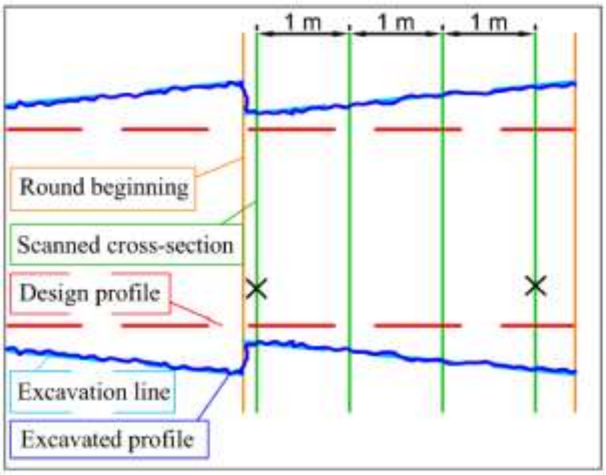

Figure 11. Round plan view. Each scanned profile (green line) is assigned to its round, using beginning and ending round KP. First and last profiles (orange lines) of each round were discarded because their blasting results can be affected by drilling operations or blast results.

The overbreak has been evaluated based on distances and areas. The overbreak is defined in two ways:

- Over-excavation: it is the extra void outside the design contour line. It is evaluated as positive overbreak.
- Under-excavation: it is the void inside the design contour line. It is evaluated as negative overbreak.

This distinction is necessary for the correct evaluation of the overbreak. In fact, the over-excavation affects the shotcrete thickness, the rock support and the mucking; the under-excavation is not admitted in the contracts and it affects the scaling.

Figure 12 resumes in four steps the data treatment and matching.

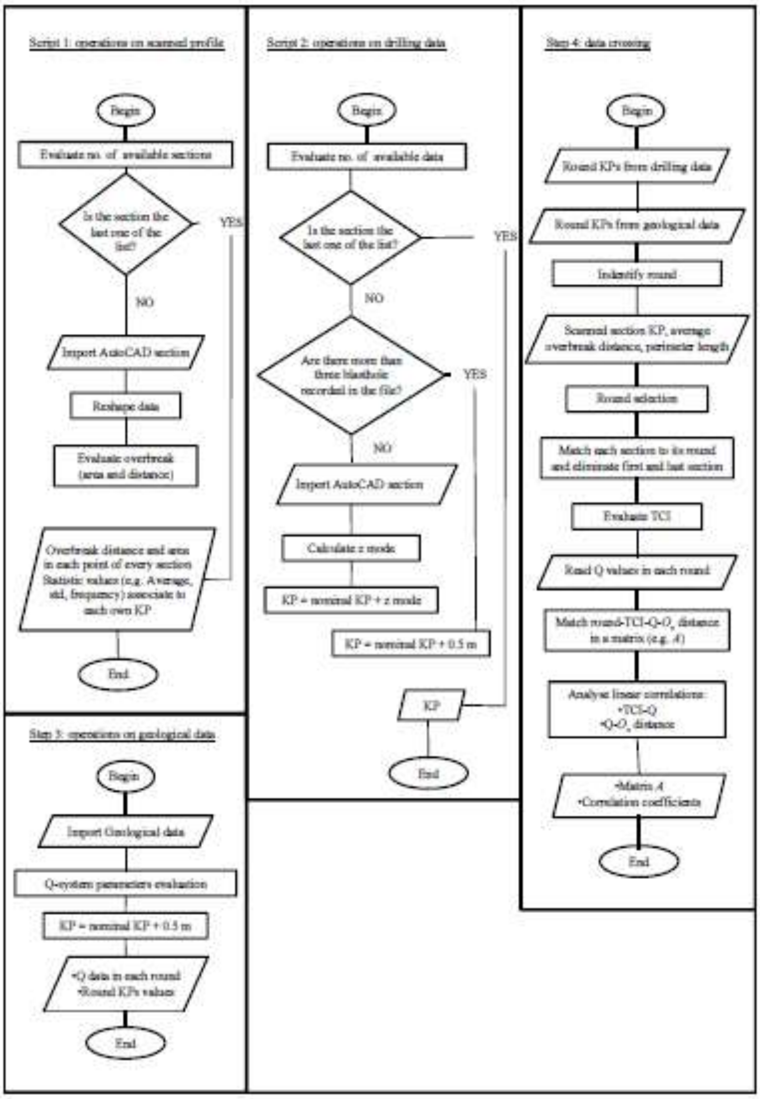

Figure 12. Flow chart of the first three step of the analysis. Step 1 elaborates data from profile survey, working from *.dxf* files and return a structure that contain, among others, statistical evaluations on overbreak (that are used after). Step 2 works on geological data (from *.xls* file*)* and evaluates Q system factors. Step 3 calibrates round beginning-end kilometric point proceeding from drilling data (*.MWD*). Step 4 matches each scanned profile to its own round and calculate TCI. It also evaluates the correlation TCI - Q value and over-excavation - Q in each round.

In this paper, cross-section contour have been analysed on the 65% upper part as shown in Figure 13. This choice depends on the low quality of the floor and corner profile survey: these bad data corrupt the results showing an unrealistic under-excavation, but the horizontal cut erases their contribution in almost every section.

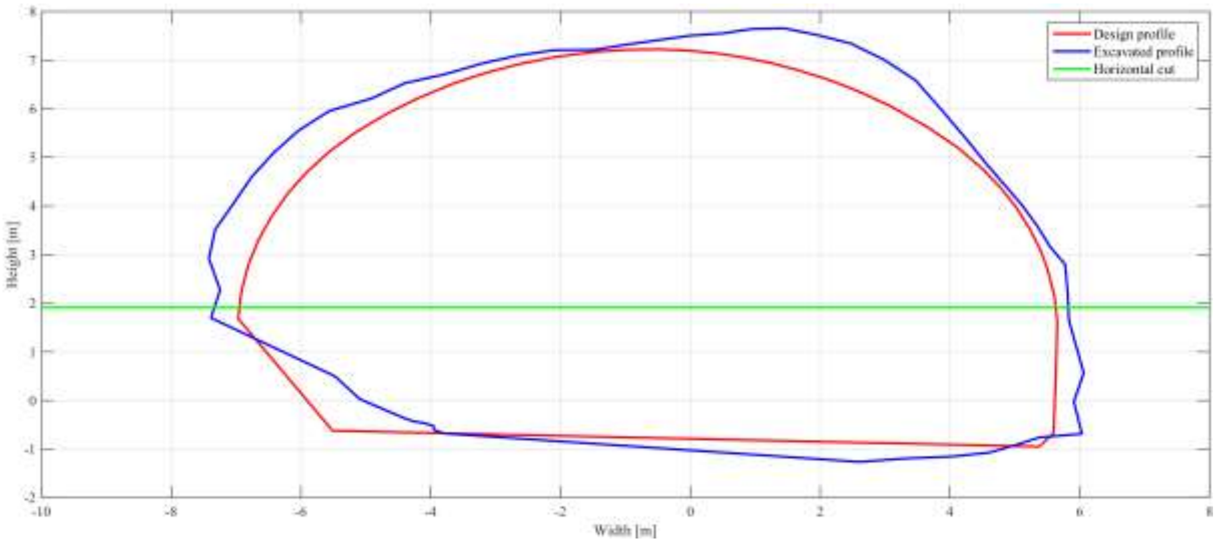

Figure 13. Section KP = 5799. The horizontal line is set at 35% of tunnel height to keep out the corner of the cross-section. All the data about over-excavation that are used for the results and discussion comes from the contour above the horizontal cut. In this way the most of profile scanning errors are not considered.

5. Results and discussion

The automatic analysis mentioned in the previous section collects, selects and treats a huge number of data. This section presents the results validated in a case history focusing on the overbreak (both in area and distance evaluation), trying to correlate them with the Q-system classification. Based on available data and on Equations 4a and 4b, TCI values are consequently processed. This index could allow time and costs reduction because it mainly uses profile scanning data that can be collected and analysed automatically. Anyway, to make its use easy and efficient, an extensive casuistry is required to build a TCI-value classification. Data reported in this study can improve the database and the evaluations required to build a related classification.

As said, all the evaluations on cross-section contour have been done on the 65% upper part (hereafter no more specified), in order to avoid survey inaccuracy. In fact, the survey was probably affected by the presence of muck left in place, disturbance due to ventilation system or water particles that reflected the laser ray in a wrong way (*Gikas*, 2012).

5.1 Tunnel quality indicator: overbreak

The beginning of the round corresponds to minimum peaks of the excavated area. In facts the end of each round must be larger than round beginning due to the drilling lookout. This trend is confirmed as shown in Figure 14.

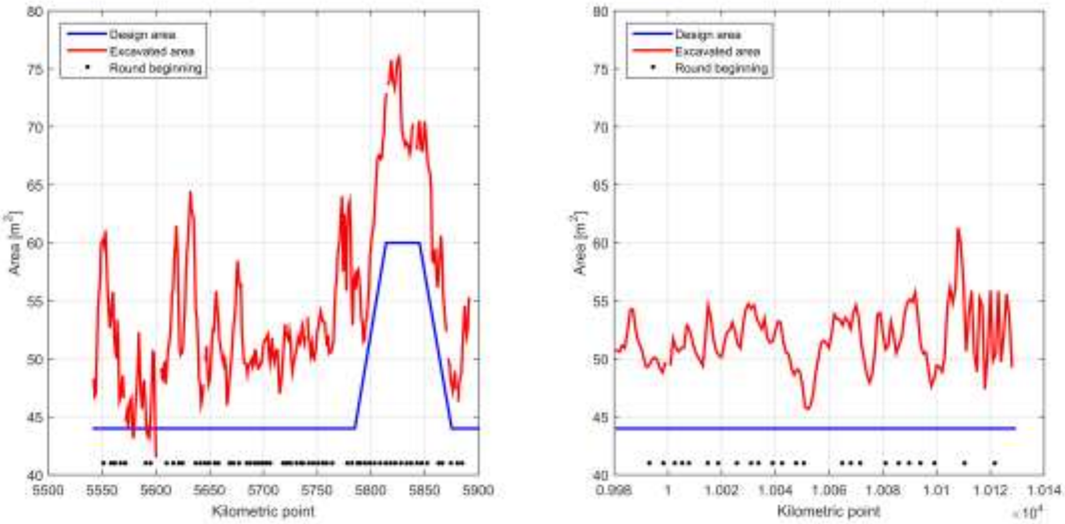

Figure 14. (left) Areas of the 65% upper zone of the cross section surveyed from Portal 1. (right) Areas of the 65% upper zone of the cross section surveyed from Portal 2. Blue line shows the design area above the horizontal cut and interpolates the theoretical values in the increasing/decreasing segments. Black points show the beginning KP of the analysed rounds.

Most of the cross-sections do not present under-excavation, as it should be. Anyway some exceptions, especially section from KP 5570 to KP 5603 and from KP 10004 to KP 10013 present under-excavation due to errors during the contour scanning. However, the actual area is still bigger than the design one because the over-excavation compensates the under-excavation area. Anyway, the following results concern the over-excavation and omit any deeper consideration on under-excavation because, as said, it is not admissible and affects scaling costs.

The over-excavation is measured as distance during the surveys but the over-excavation additional costs depend on volume of additional muck and voids that needs to be shotcreted. These volumes can be calculated a posteriori from the over-excavation area of each cross-section. In this case study, the developed code measures over-excavation area (that correspond to a volume for unit of advance - 1 m) from polygons between design and actual contour but it can be correlated to the average over-excavation distance and the design contour length, as shown in Figure 15. The two values are linearly correlated: 1.50 slope in a range of [1.45; 1.55], 0.20 intercept in a range of [-0.09; 0.50], 0.88 of R^2 value.

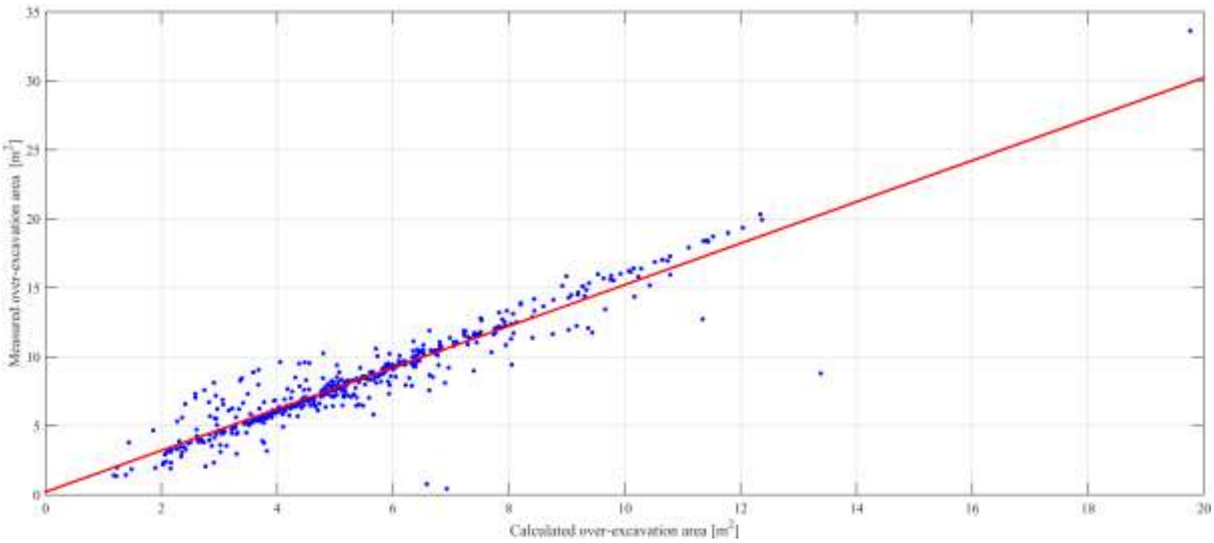

Figure 15. Over-excavation area can be calculated as function of average overbreak distance ($\bar{O}_v$) and design contour length (D_l), using the relation $O_{v_{calculated\ area}} = \bar{O}_v \times D_l$. This relation allows to consider only distance values.

This relation allows to work only on over-excavation distance, as most of the literature does. However, in some cases (*Mahtab*, 1997; *Mandal and Singh*, 2009; *Olsson*, 2010) the over-excavation normalized area (as shown in Equation 2) is used. In this case study the average value of the over-excavated ratio is quite similar for the sections from Portal 1 (18.5%) and from Portal 2 (17.1%). Considering all section from both portals, the value is 18.1±7.7 % (mean and standard deviation), in a range between 1% and 46%. Values from literature are in a range between 7.3% up to 51.9% (*Mahtab*, 1997; *Mandal and Singh*, 2009; *Olsson*, 2010) and *Olsson* (2010) proposes an admissible overbreak ratio of 25% as upper limit.

According to the Norwegian regulation the admissible over-excavation (O_v) is the minimum value between 0.4 m and the value calculated as function of cross-section area (Equation 3, Table 5). A third limit is interpolated and used to evaluate all the cross-section in which the area progressively increases/decreases due to switch between theoretical section T9.5 and T12.5. No references for these sections were found in literature.

Table 5. Maximum admitted O_v depending on cross-section area and evaluation of case study available sections.

Theoretical cross-section	Area [m²]	Maximum admissible O_v [m]	No. of sections	Compliant sections (% on their group)
T9.5	74	0.60	400	84
T12.5	100	0.70	28	79
Interpolation	[74; 100]	0.65	59	92

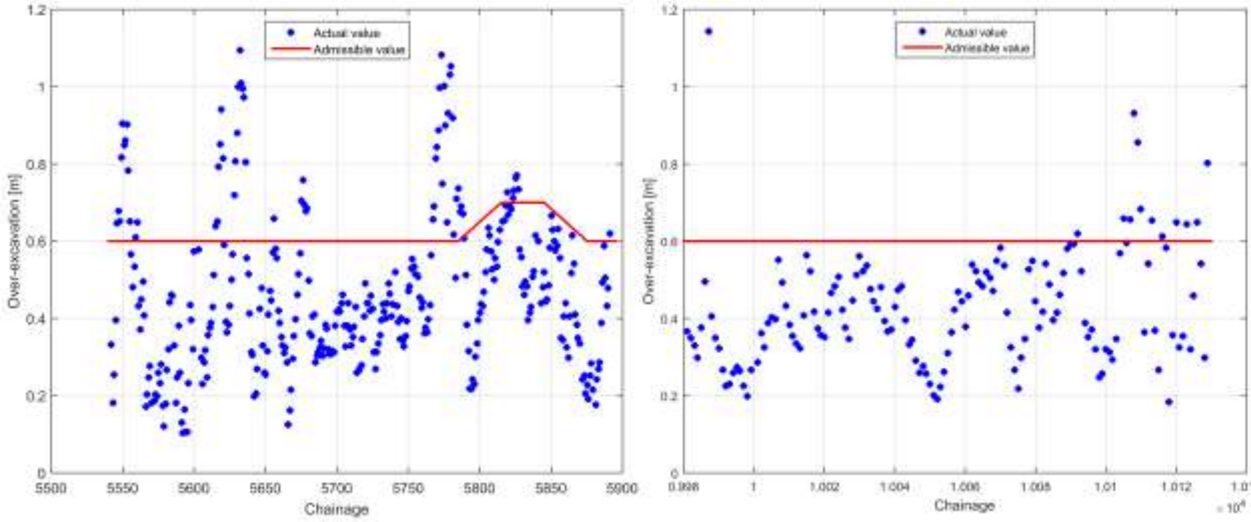

Figure shows the average value of the over-excavation distance for each scanned section and the threshold values. Average values of over-excavation of each cross-section from both Portals are considered to obtain average, maximum and minimum values for the entire tunnel (Table 6).

Table 6. Evaluation on overbreak distances.

Values on sections from Portal 1 and Portal 2	O_v
Average [m]	0.46
Standard deviation [m]	0.19
Maximum [m]	1.10
Minimum [m]	0.10

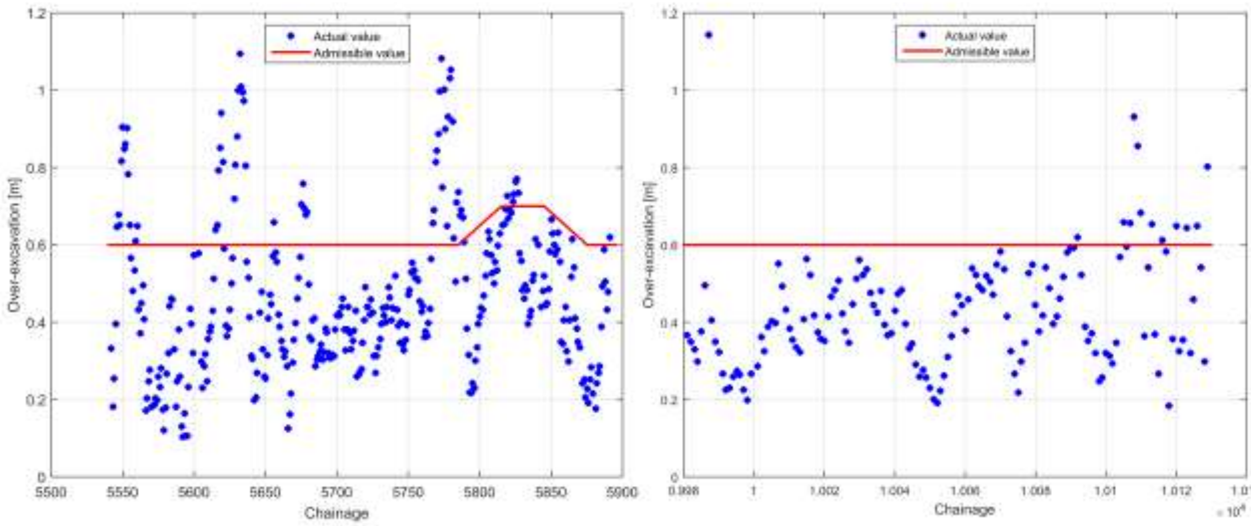

Figure 16. (left) Average over-excavation distance in each section from Portal 1 compared with admissible values. (right) Average over-excavation distance in each section from Portal 2 compared with admissible values.

5.2 Correlation between geological and topographical survey

The main values for all the Q-system parameters suggest a fair-good rock (*RQD*) with an irregular and smooth undulating surface (J_r), usually two or three joint sets were surveyed plus some random joint (J_n) and the most of these joint were slightly altered (J_a); the rock mass is medium stressed (*SFR*). The surveyed rock mass is dry, and the value J_w is constantly set on 1. As summary, the rock mass can be considered very poor - poor - fair (classes IV, V, VI) quality all along the tunnel.

Under the assumption that every drill an blast cycle was done with the same technique and the same equipment, available geology data are compared with overbreak distances.

Figure 17 shows Q progression along the tunnel axis and overlaps average over–excavation of each scanned profile.

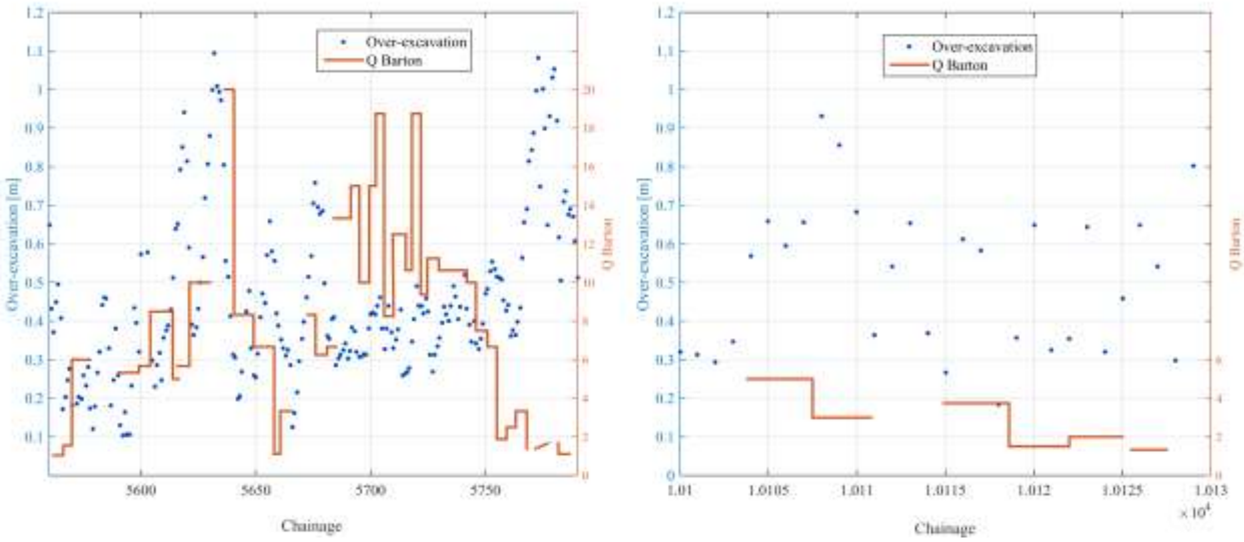

Figure 17. (left) Over-excavation distance and Q of each scanned or surveyed section from Portal 1. (right) Over-excavation distance and Q of each scanned or surveyed section from Portal 2.

Average over-excavation of each round is compared with the Q value trying to demonstrate a relation between Q index and excavation results. 18 rounds present enough data to be analysed in this case; no linear correlation was found between the average over-excavation distance and Q values. Table 7 shows the range of over-excavation distances for each Q classes. These ranges are similar for all the classes and they cannot be used to predict the blasting results in each round.

This lack of correlation could depend on shotcreting phase, as it modifies in sense of smoothing the contour profile. In fact, best rock mass quality should require less shotcrete (and the opposite on low quality surfaces), thus over-excavation results are emphasised (or mitigated). But available data do not allow confirming this hypothesis, thus shotcreting thickness has been considered as a relatively constant value.

Table 7. Over-excavation ranges in Q classes.

	Over-excavation [m]			
	Min	Max	Mean	Std
Q ≤4	0.35	1.50	0.75	0.36
4<Q≤10	0.16	1.23	0.64	0.26
10< Q ≤40	0.42	0.94	0.64	0.20

5.3 Tunnel quality indicator: Tunnel Contour Quality Index (TCI)

The Tunnel Contour Quality Index - TCI (3.3) can be evaluated for each round.

Equation 4b has been used, following the procedure well explained in *Kim* (2009) for the coefficient determination. Table 8 shows the values used in this case study to calculate TCI in each round and for the whole tunnel.

Table 8. Values of TCI coefficients and weight.

Coefficient of adjustment		Weight[a]	
C_1	0.0049	W_1	4.5
C_2	0.6765	W_2	4.5
C_3	0.0206	W_3	1.0
C_r[a]	300		

a) values obtained by *Kim* (2009)
b) values calculated by using this case data and following Equation 5

TCI obtained values vary from 40 up to 86; the two Portals do not present different ranges of this value, and they can be analysed together (Figure).

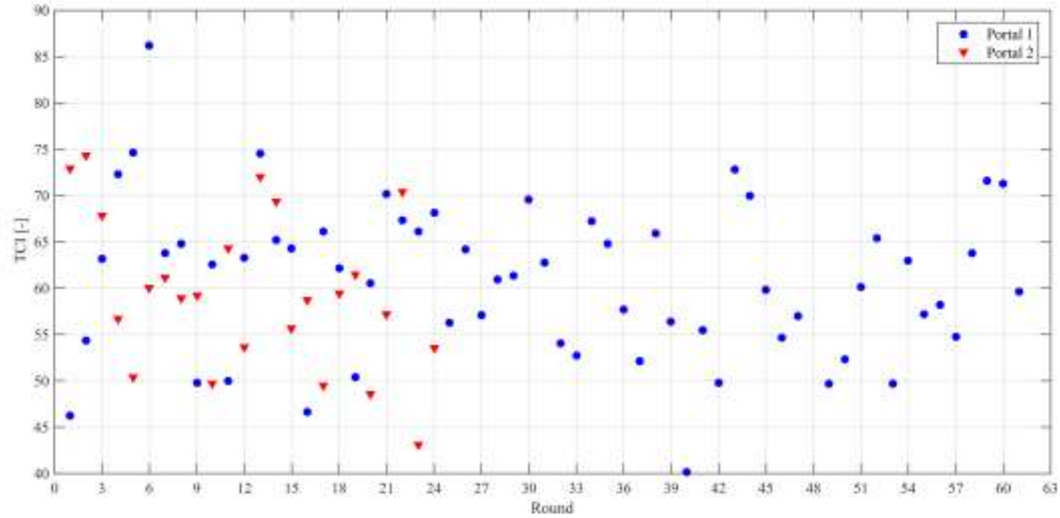

Figure 18. TCI in each round from Portal 1 and Portal 2. The most of the values are between 45 and 75 in both Portals.

The distribution of round TCI is shown in Figure 9 according to the classes that were proposed in literature (*Kim*, 2009). The largest frequency is found between 62 and 65; the whole range is between 38 and 77. The case study presents good round TCI compared with literature case studies (Table 9).

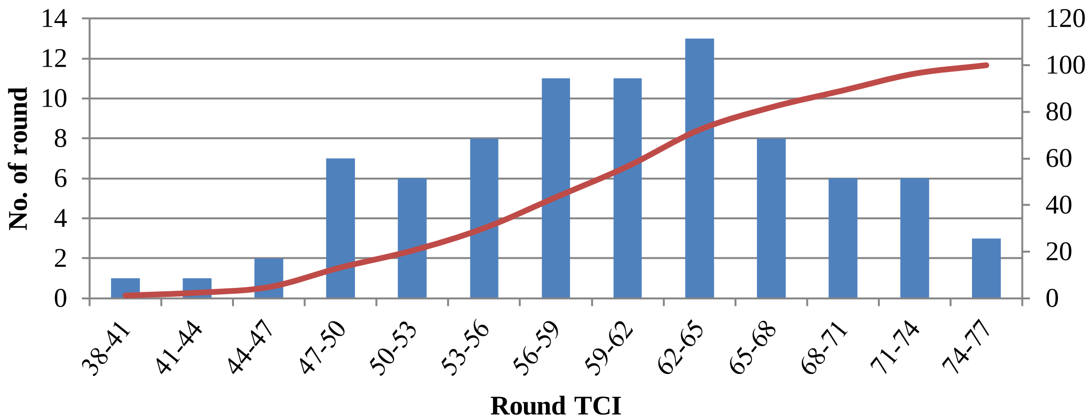

Figure 19. Distribution and cumulative distribution of round TCI based on case study data.

Table 9. Comparison with round TCI results from literature (*Kim*, 2009; *Kim and Bruland*, 2010) and case study tunnel.

Tunnel	Minimum round TCI	Maximum round TCI	Largest frequency
LS02 (Norway)	38	62	47-50
Marienborg (Norway)	47	58	53-56

Misiryung (Korea)	56	75	62-71
Case study	38	77	62-65

TCI can be evaluated on the whole tunnel if more than five rounds are available (*Kim*, 2009). Table 10 shows the values of the index parameters in the case study, comparing the resultant TCI with literature examples (*Kim*, 2009; *Kim and Bruland*, 2010; *Kim and Bruland*, 2015). The TCI value shows a good tunnel contour quality, quite near to the result in Misiryung tunnel that was considered to have a very good excavation result.

Table 10. Comparison of TCI in different studies.

TCI parameter	Case study
C_1	0.0049
C_2	0.6765
C_3	0.0206
Average O_v [cm]	70.2
Average RCL	1.2
V_0	5.4
TCI	**57.4**
Tunnel TCI from literature	
LS02 (four segments)	48.2
Marienborg	52.1
Misiryung	62.4

6. Conclusions

In this paper, tunnel contour evaluation is obtained in terms of overbreak and related to rock mass conditions. Over-excavation affects timing and costs (i.e. mucking and concrete/shotcrete costs), because the additional excavated volume usually needs to be replaced by additional shotcreting and other reinforcing works. In order to achieve a reliable procedure, the quality of almost 500 m of a roadway tunnel has been evaluated. For this purpose, measurements from three different sources are automatically processed: topographical measurements of excavated contour, geological mapping of the rounds and Measuring While Drilling (MWD) data.

According to the discussion, the work is related with the analysis of resulting profiles from D&B tunnelling and has provided the following main results:

1. A detailed analysis of large data sets is only possible by processing automatically the data. A code developed in Matlab environment has been created to quantify the overbreak caused by blast; it processes automatically MWD, geological and topographic data and stores them in the same numerical format. This code can be easily adapted to other case studies.
2. Results from the analysis are mainly focused on the characteristics of the excavated contour in comparison with the theoretical section, considering over-excavation in terms of area and distance separately and demonstrates that they are close linearly correlated. The study case exhibits an over-excavated distance of 0.46±0.19 m (mean and standard deviation). This value is in general under the admissible Norwegian limit of 0.6 up to 0.7 m calculated on the theoretical area.
3. The quality of data in the study case allows to focus only on geological causes and Q index is adopted to describe rock mass conditions. Q index exhibit from a very poor to fair quality. No strong numerical correlation between Q and over-excavation has been found. This result probably suggests that drill operations and blast design influence are stronger on contour quality control.
4. Tunnel quality has been also evaluated using the engineering index TCI. In the case study TCI has a value of 60.5 ± 8.4; the largest frequency is found between 62 and 65; the whole range between 38 and 77. These values are relatively well with those for other tunnels in literature. As more than five rounds are available, the index can be evaluated for the entire tunnel: overall TCI value is 57.4 and shows a quite good quality of the excavation (if compared with the studies of the index developer).

Further works based on the application of TCI to other case studies could enlarge the records and validate the efficiency of the index itself. The impact of shotcrete thickness should be deeply evaluated when the survey is done after shotcreting (as in this case) and more parameters should be recorded during the blast (e.g. PPV) in order to calculate other damage indices (e.g. BDI) and find out the most representative in terms of correlation with over-excavation. In the field of engineering application, the TCI index can provide a tool to evaluate tunnel contour quality in a unique way, simplifying the contractual requirements.

7. Acknowledgements

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