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TBM Tunnelling in Faulted and Folded Rocks

Construction mécanique de tunnels dans des roches faillées et plissées

TBM Vortrieb in gestörtem und gefaltetem Gebirge

Ecole Polytechnique Fédérale de Lausanne EPFL
Laboratoire de Mécanique des Roches LMR

Erika Paltrinieri
Prof. Jian Zhao

Projet de recherche FGU 2007/004 sur demande de Arbeitsgruppe Tunnelforschung (AGT)

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Résumé

Le but de ce projet est l'étude des performances des tunneliers, en particulier des TBM à gripper, dans les terrains très fracturés et dans les failles.

Le développement rapide des tunneliers en milieux rocheux est principalement dû à leurs avantages par rapport aux méthodes d'excavation traditionnelles : avancement en continu, meilleure sécurité et vitesse accrue dans des conditions géologiques favorables. Mais dès que ces conditions se détériorent, comme c'est le cas dans des zones très fracturées et perturbées, l'avancement peut être fortement freiné, voire stoppé.

Les failles représentent un danger majeur lors de la construction d'ouvrages souterrains, principalement pour les instabilités du front qu'elles occasionnent. Ce rapport décrit d'abord les principaux défis occasionnés par ces conditions géologiques difficiles, ainsi que les problèmes géotechniques majeurs tels qu'ils ressortent d'études de cas. Puis il présente succinctement les modèles les plus courants de prédiction des performances des tunneliers dans des conditions optimales, ceci afin d'identifier les principaux paramètres utilisés.

L'excavation dans des zones de failles est généralement associée à une forte anisotropie des déplacements des parois et à une redistribution secondaire des contraintes autour de la cavité durant et après l'excavation. Ceci conduit souvent à de longues interruptions de l'avancement et une forte réduction des opérations. L'hétérogénéité de ces zones complique leur caractérisation géotechnique : il s'agit en réalité d'une succession de parties faibles et de parties plus résistantes. Pour analyser ce comportement particulier, une étude bibliographique de la classification géologique et géomécanique des roches cataclastiques a été menée.

Afin de pouvoir analyser la relation entre les conditions géologiques difficiles et les performances des TBMs, des données sur plusieurs tunnels ont été récoltées et stockées dans une base de données « performances de tunneliers ». Y figurent à la fois les caractéristiques du tunnel, les paramètres et performances du tunnelier, ainsi que les paramètres géologiques et géotechniques de la roche intacte et du massif rocheux. Même s'il est difficile d'obtenir des données complètes et détaillées, cette base a l'avantage de réunir des données provenant du terrain, de laboratoires et de la littérature.

Une étude préliminaire a été conduite pour analyser des corrélations entre les paramètres de performance des tunneliers (en particulier la pénétration et l'avancement) et les géo-paramètres utilisés habituellement dans les modèles de prédiction des performances, comme la résistance de la roche et le degré de fracturation. Afin de tenir compte de la tectonisation et de l'altération du terrain, un nouveau paramètre de résistance de la masse rocheuse (appelé UCS_H) a été considéré. Celui-ci représente en fait une réduction de la résistance de la roche en fonction du „Geological Strength Index" (GSI). Mais même si ces valeurs de résistance représentent mieux les conditions réelles, les corrélations avec les paramètres des tunneliers restent insatisfaisantes. La principale raison en est que les conditions réelles du terrain sont trop complexes pour pouvoir être décrites par un seul facteur. Finalement, pour introduire une approche plus globale du comportement de masses rocheuses, de possibles corrélations entre des systèmes de classifications, comme le RMR, et les performances ont aussi été investiguées. Même si quelques tendances ont pu être observées, la grande dispersion des résultats a confirmé qu'il était difficile de prédire les performances de tunneliers dans des géologies complexes à l'aide de modèles préexistants, établis pour l'excavation dans de bonnes conditions géologiques.

L'étape suivante a été le développement d'un nouveau système de classification des masses rocheuses très fracturées et des failles à l'aide de paramètres géomécaniques pertinents pour une description complète de zones perturbées. Quatre classes de « zones perturbées » ont pu être identifiées en se basant sur les degrés de fracturation et d'altération, décrits principalement par le nombre de joints par mètre cube (J_v) et par

l'indice GSI. Puis la base de données « performances de tunneliers » a été analysée à l'aide de ces classes. Pour chacune des classes, on a observé une réduction des paramètres sélectionnés (taux de pénétration, vitesse de rotation de la tête et avancement journalier) par rapport aux valeurs obtenues dans de bonnes conditions géologiques. Les plus grandes réductions ont été constatées dans les classes décrivant les plus mauvaises conditions géologiques. Les résultats de ces analyses sont très intéressants, car ils permettent de quantifier l'effet de la dégradation des conditions géologiques sur les performances des tunneliers, aussi bien en termes de taux de pénétration que de vitesses d'avancement.

Ces résultats ont ensuite été utilisés pour faire des analyses probabilistes des temps et coûts de construction de tunnels avec le logiciel « DAT » (Decision Aids for Tunnelling). Ce logiciel permet en effet de quantifier l'influence des incertitudes liées à la géologie et de celles liées à la méthode de construction sur la distribution des temps et coûts de construction. Dans le cas présent, les DAT ont été utilisés pour estimer l'effet de conditions géologiques particulièrement difficiles, comme p. ex. la présence de failles, sur les coûts et les durées des opérations d'un tunnelier. Pour cela, l'excavation a été simulée à travers différents profils géologiques, où l'alternance des conditions géologiques a été décrite à l'aide des classes de « zones perturbées », la vitesse dans chacune des quatre classes correspondant au taux de réduction obtenu précédemment. L'analyse a conduit à une estimation fiable de l'influence de conditions géologiques dégradées (de roche fortement fracturée, en passant par des masses perturbées, jusqu'à un matériau broyé) sur les temps et coûts de construction.

Les résultats de cette recherche permettent de mieux comprendre comment de mauvaises conditions géologiques réduisent les performances de tunneliers par rapport aux conditions optimales. Ils montrent aussi le besoin d'une meilleure caractérisation géomécanique des masses rocheuses fortement fracturées et perturbées. Pour ceci, il faudrait considérer des paramètres spécifiques, en lieu et place de ceux utilisés actuellement pour estimer les performances dans de bonnes roches.

MOTS CLÉS :

Roche de mauvaise qualité; classification des zones perturbées; prévision de la performance de tunneliers; base de données; DAT.

Zusammenfassung

Ziel dieses Projektes ist es, die TBM-Leistungen (insbesondere die der Gripper-TBMs) in stark zerklüftetem Gebirge und in Störzonen zu untersuchen.

Die rasche Entwicklung der TBMs bei Felstunneln ist hauptsächlich durch deren Vorteile gegenüber den konventionellen Ausbruchsmethoden, wie z.B. kontinuierliche Operationen, sicherere Arbeitsbedingungen und höhere Vortriebsleistungen, zurückzuführen. Trotz der ausgezeichneten Leistungsmöglichkeiten in günstigen Gebirgsverhältnissen, kann der TBM-Vortrieb jedoch in ungünstigeren Geologien, wie in sehr zerklüfteten und gestörten Zonen, stark verlangsamt, oder sogar gestoppt werden.

Im Untertagebau stellen Störzonen eine der grössten Gefahren dar, insbesondere was die Tunnelbrust-Instabilitäten anbelangt. Dieser Bericht beschreibt zuerst die Tunnelbau-Herausforderungen in diesen schwierigen Gebirgsverhältnissen, sowie die in bekannten Fallstudien geschilderten geotechnischen Hauptprobleme. Dann werden die geläufigsten Modelle zur Vorhersage der TBM-Leistungen rasch vorgestellt und die Hauptparameter zur Bestimmung der TBM-Leistungen in optimalen Bedingungen aufgelistet.

Der Tunnelbau in Störzonen ist meistens mit stark anisotropen Tunnelwandverformungen und mit sekundären Spannungsumlagerungen um den Hohlraum während und nach dem Ausbruch verbunden. Dies führt oft zu langen Vortriebsunterbrüchen mit stark reduzierten TBM-Operationsparametern. Wegen deren Heterogenität ist die geomechanische Charakterisierung dieser Zonen sehr schwierig: in Wirklichkeit sind sie aus einer Abwechslung von schwachen und festen Gesteinskomponenten gebildet. Um dieses besondere Verhalten zu untersuchen wurde ein Literaturstudium über geologisch/geomechanische Klassifikationen von Störzonengestein, generell als „kataklastisches“ Gestein bekannt, durchgeführt.

Um die möglichen Zusammenhänge zwischen schwierigen Gebirgsverhältnissen und TBM-Leistungen aufzudecken, wurden Daten über mehrere Tunnel gesammelt und in eine TBM-Leistungsdatei abgespeichert. Diese ist in zwei aufgeteilt: der erste Teil beinhaltet die Tunnel- und die TBM-Eigenschaften sowie die TBM-Leistungs-Parameter, und der zweite Teil die geologisch-geotechnischen Parameter des intakten Felsen und der Felsmassen. Trotz den Schwierigkeiten, komplette TBM-Daten und detaillierte geologische Informationen zu erhalten, hat diese Datei den Vorteil, in-situ-Werte, Laborresultate und Angaben aus der Literatur zu vereinigen.

In Voruntersuchungen wurden Korrelationen zwischen TBM-Leistungsparametern (nämlich Penetrations- und Leistungsraten) und Gebirgsparametern (wie Festigkeit und Trenngrad), welche allgemein in TBM-Leistungsmodellen benützt werden, untersucht. Um den Tektonisierungs- und den Verwittterungsgrad der Felsmasse mit zu berücksichtigen, wurde ein neuer Felsfestigkeitsparameter, der sogenannte UCSH, der die Materialfestigkeit anhand des „Geological Strength Index” (GSI) reduziert, eingesetzt. Auch wenn die so erhaltenen Festigkeitswerte an Relevanz gewinnen, bleiben die Korrelationen mit den TBM-Leistungen jedoch unbefriedigend. Grund dafür ist, dass die eigentlichen Gebirgsverhältnisse zu Komplex sind, um mit einem einzigen Faktor erfasst werden zu können. Um die Felsmassen globaler zu charakterisieren, wurden auch mögliche Korrelationen zwischen bekannten Klassifikations-Systemen, wie z.B. RMR, und TBM-Leistungen untersucht. Manche Trends konnten zwar identifiziert werden; die sehr gestreuten Resultate bestätigten jedoch die Schwierigkeiten einer Vorhersage der TBM-Leistungen in komplexen geologischen Verhältnissen anhand von existierenden Modellen und Charakteristiken, die für gesunden Fels entwickelt worden sind.

Als nächster Schritt wurde dann ein eigenes Klassifikationssystem für stark zerklüftete Felsmassen und Störzonen mit relevanten geomechanischen Parametern zur vollständigeren Beschreibung von gestörten Zonen entwickelt. Identifiziert wurden vier „Störzonen-Klassen“, welche den Zerklüftungs- und Verwitterungsgrad der Felsmassen, charakterisiert durch die volumetrische Kluftanzahl (Jv) und das GSI, kennzeichnen.

Dann erfolgte die Analyse der in der Datei abgespeicherten TBM-Leistungen nach der neuen Klassifikation. In jeder der vier Klassen wurde eine Reduktion der TBM-Leistungsparameter (Penetrationsrate, Bohrkopfdrehgeschwindigkeit und tägliche Vortriebsleistung) gegenüber den Bestwerten in guten Gebirgsverhältnissen festgestellt. Je ungünstiger die Klasse, desto grösser war die Reduktion. Diese Resultate sind von höchster Relevanz, da sie es tatsächlich erlauben, die Auswirkung von degradierten Felsmassenverhältnissen auf die TBM-Leistungen (sowohl auf die Penetrationsraten, wie auf die Vortriebsleistungen) zu quantifizieren.

Mit diesen Ergebnissen wurden dann Simulationen mit den "Decision Aids for Tunnelling" (DAT) – Entscheidungshilfe für den Tunnelbau (EHT) auf Deutsch - durchgeführt, um probabilistische Tunnelbaukosten und –Zeiten zu berechnen. Mit der Software EHT kann nämlich generell der Einfluss der geologisch/geotechnischen und der baubedingten Unsicherheiten auf die Baukosten und –Zeiten quantifiziert werden. Im vorliegenden Fall wurden die EHT dazu benützt, die Auswirkungen von schwierigen Gebirgsverhältnissen, wie z.B. von Störzonen, auf die Kosten und Dauern der TBM-Operationen abzuschätzen. Dazu wurde der Tunnelbau durch mehrere geologische Profile simuliert, wobei die abwechselnden Felsverhältnisse mit den vier „Störzonen-Klassen" beschrieben wurden. Input waren die zuvor für jede Klasse erhaltenen Vortriebsleistungs-Reduktionen. Die Analyse führte zu einer zuverlässigen Abschätzung des Einflusses von degradierten geologischen Verhältnissen (von stark zerklüftetem Fels über zerstörten Fels bis zu zermahlenem Material) auf die Baukosten und –Zeiten.

Diese Forschungsarbeit gibt nützliche Einsichten in die TBM-Leistungsminderungen, welche in schlechten Tunnelbauverhältnissen eintreten können. Sie zeigt auch das Bedürfnis nach einer besseren geomechanischen Charakterisierung der stark zerklüfteten und gestörten Felsmassen, und zwar durch das Einsetzen von spezifischen Parametern für verwitterte und degradierte Felsmassen, statt den Parametern, die heute zur Abschätzung der TBM-Leistungen in guten Gebirgsverhältnissen dienen.

SCHLÜSSELWÖRTER:

Schwacher Fels; Störzonen-Klassifikation; Vorhersage TBM-Leistungen; Database; DAT (EHT).

Abstract

This project focuses on the study of the performance of Tunnel Boring Machines (in particular gripper TBMs) in highly jointed rock masses and fault zones.

The great development of TBMs in rock tunnelling industry is mainly due to their advantages with respect to conventional excavation, such as continuous operation, safer working conditions and greater rate of advance. However, despite the possible excellent performance of TBMs in favourable ground conditions, the advance rates could be significantly slowed down or even obstructed in limiting geological situations such as highly fractured and fault zones.

For underground constructions, a fault zone is one of the most dangerous events especially for what concerns the instability phenomena that may occur at the excavation face. After an introduction about the major issues resulting from tunnelling in these difficult ground conditions, and a description of the main geotechnical problems encountered in famous case-studies, a brief review of the most common TBM-performance prediction models is reported. The objective is trying to point out the most common parameters used for evaluating the TBM performance in optimum advancing conditions.

Tunnelling in fault zones is generally associated to strongly anisotropic wall displacements as well as a secondary stress re-distribution around the opening during and after the excavation. This often leads to long stops of the excavation face with strong reduction of the TBM operations. The geomechanical characterisation of these zones is very difficult due to their heterogeneous nature: these zones are actually a combination of weak and strong rock components. This particular behaviour has been studied through a literature review about geological and geomechanical classifications of fault rocks, generally called “cataclastic” rocks.

In order to investigate possible relationships between difficult rock mass conditions and TBM performance, the data of several tunnel projects have been collected in a specific database (TBM-performance database). This database is subdivided into two sections: the first one contains the tunnel characteristics, TBM specifications and TBM performance parameters; while the second one considers the geological-geotechnical parameters of the intact rock and rock mass. Despite the difficulties in gaining complete TBM and detailed geological information, this database compiles data obtained directly from the field, laboratory tests and literature.

Preliminary analyses have been carried out in order to identify correlations between TBM performance parameters (namely, penetration and advance rates) and rock mass parameters generally used in common TBM performance prediction models, such as rock strength and fracturing degree. Moreover, in order to take into account the tectonisation and the weathering degree of the rock materials a new rock mass strength parameter (called UCS_H) has been considered. This is done by reducing the material strength by means of the Geological Strength Index (GSI). Although the strength values become more representative of the real conditions, the correlation with the TBM parameters remains unsatisfactory. This is mainly because the considered ground conditions are very complex and cannot be described in terms of a single factor. Finally, in order to introduce a more global approach for characterising rock mass behaviour, possible correlations between rock mass classification systems (RMR) and machine performance have been also investigated. Although some trends could be identified, the scattered results confirm the difficulties in predicting the machine performance in complex geological environments starting from existing models and characterising indexes proper to good rocks.

The next step of the study has been the development of a classification system for highly fractured rock masses and fault zones. The aim of this classification is to select representative parameters that should be considered in order to obtain a more complete geomechanical description of disturbed zones. Four “fault zone” classes have been

identified considering the fracturing and the weathering degrees of the rock mass, mainly described in terms of volumetric joint count (J_v) and GSI. Then, the analyses of the data recorded in the TBM-performance database have been done according to this new classification. For each class a reduction rate of the selected TBM-performance parameters (penetration rate, cutterhead rotation speed and daily advance rate) has been observed with respect to the best tunnelling performance recorded in good ground conditions. This reduction proves to be more evident in the classes describing the worst tunnelling conditions. The results of these analyses are of great relevance as they allow to effectively quantify the effect of degrading rock mass conditions on the TBM advance (in terms of both penetration and daily advance rates).

The results obtained in the previous steps have been used to carry out probabilistic analyses of the tunnel construction time and costs by means of the “Decision Aids for Tunnelling” (DAT). The DAT are numerical tools that can be used to evaluate the influence of the uncertainties related to both the geological/geotechnical conditions and the construction process on final tunnel construction time and costs. In this framework, the DAT have been used for assessing the effects that demanding ground conditions, such as fault zones, may have on tunnelling operations in terms of costs and delays. This is done by simulating the tunnel construction in several geological profiles, where changing rock mass conditions are described in terms of the four “fault zone” classes. The advance rate reductions, previously obtained for each class, have been the input for the simulations. The outcomes of the analyses give a reliable estimation of the effect of degrading geological conditions (from highly fractured rocks, to faulted rocks, to crushed material) on the final tunnel construction time and costs.

The results of this research may provide useful insights regarding the reduction of the machine performance that can take place in bad grounds, with respect to an estimated best performance in good tunnelling conditions. Moreover, the study highlights the need to better characterise, from a geomechanical point of view, the highly fractured and faulted rocks. This could be done by considering the parameters more representative of the altered/degraded state of the rock mass, instead of the parameters which are today considered as a reference for estimating TBM performance in good rocks.

KEYWORDS:

Weak rocks; Fault zone classification; TBM-performance prediction; Database; DAT.

1 Tunnelling in weakness zone

Tunnelling activities are increasing enormously in these last years and mechanised excavation is catching on more and more as alternative to the conventional technology based on drilling and blasting. The great development of Tunnel Boring Machines (TBMs) in tunnelling industry is mainly due to its advantages, with respect to conventional excavation method, such as continuous operation, safer working conditions, reduced damage at ground surface and grater rate of advance. However, the TBM performance could be significantly reduced in changing ground conditions, because of the lack of versatility of the machine, and in particular geological situations like very strong or very poor rock masses, where lower advance rates are generally recorded.

One of the most problematic events that may occur in TBM tunnelling (especially for what concerns gripper TBMs) is the crossing of weakness zones. Although the presence of important fault zones is generally identified along the tunnel alignment in the project phase, in some cases it may still represent an unexpected incident due to either a lack of warning during excavation or an underestimation of the problem seriousness [1]. Tunnelling in such difficult ground conditions could cause an increase in the project cost (in terms of additional ground improvements and consequent substantial delays) and plays a central role for the safe completion of the works. An understanding of the geological development and the occurrence of weakness zones is therefore important in the planning and construction of a project, although they generally occur along only 1-15% of a tunnel [2].

1.1 Major issues of tunnelling in fault zones

Weakness ground conditions, besides referring to weak (sandstone, siltstone, shale, mudstone, marl and chalk, tuff, agglomerate) and weathered (hydrothermal and chemical) rocks of all types, could be caused by jointing and shear zones or faults present in the rock mass [3].

As reported by several authors [4-8], the most common geotechnical problems resulting from tunnelling in faulted/highly fractured zones are:

- instability of the excavation face;
- excessive overbreak of the tunnel contour;
- excessive deformation when squeezing ground is encountered;
- frequent changes (i.e. both magnitudes and direction) of stresses and displacements;
- large water inflows.

The possible instability phenomena (such as collapses, ravelling, slabbing, softening and slaking, swelling, squeezing and flowing ground) are generally due to the low strength and to the high deformability of the involved weak and weathered rocks, furthermore, they can be influenced by groundwater conditions and by discontinuities (bedding planes, joints, foliation and faults) in the rock mass [3].

The tunnel collapse is a sudden cave-in inwards which could interest the face, the side-walls and the lining (if occurring mainly after excavation). These face and wall instabilities in hard rock are characterised by different causes and failure mechanisms respect to the ones in soft and weak ground, where a leading role is played by the permeability.

A gradual overbreak at tunnel roof and/or walls could be associated to ravelling ground that implies a time-dependent loss of strength of the weathered rocks [9]. Significant ravelling could develop because of the combined effect of stress slabbing in a thinly-bedded rock formation and loosening along steeply dipping joints orientated nearly parallel to the tunnel [3]. The ravelling behaviour considerably reduces the rock mass stand up time, mainly depending on the degree of weathering [9].

Stress slabbing consists of detached slabs and block of rock that can fall into the tunnel excavation, unless timely support is provided [3]. The term “slabbing” is often associated to the spalling phenomenon, caused by overstressing of massive brittle rocks, that involves the creation of boundary-parallel extension fractures around tunnel opening [6]. As reported by Palmstrom [10], several authors [11, 12] define spalling as any “drop off of spalls or slabs of rock from tunnel surface several hours or weeks after blasting”. For rocks to spall, they must be more prone to tensile cracking than to the development of shear plane and shear fractures; this is the case of brittle massive rocks rather than weak and altered rocks [6]. As a matter of fact, according to Diederichs [13], spalling is the dominant process for hard rocks at depth characterised by a Rock Mass Rating (RMR) greater than 75, while RMR 60 is a reasonable lower bound value for the process to be likely [6].

One of the worst geological hazards of tunnelling in weak rocks is represented by flowing ground in which unconsolidated and saturated material can enter the tunnel flowing for great distances and for a significant time. It is generally associated to fault zones and crushed zones, where a great amount of available water at high pressure must be present and the rock must be characterised by no or low cohesion [9].

Softening and slaking are typical of rocks with high clay content that are prone to moisture changes. Softening represents a significant loss of strength and slaking results in disintegration of the rock [3]. For tunnels, slaking means problems in stand-up time, overbreak, construction and support procedures required to minimise deterioration, and it contributes to time-dependent instability of large blocks progressing along surfaces of fractures behind rock slabs [14].

Another important issue linked to fine-grained rocks is the swelling behaviour, which consists of a volume increase of the rock mass surrounding the tunnel due to the adsorption of water by clay minerals like montmorillonite or smectite. Stress-induced cracking may facilities access of water promoting swelling that “can also occur due to chemical or hydrothermal alteration of minerals such as feldspar, which produces montomorillonite, or due to the hydration of anhydrite, which produces gypsum” [3].

Very common time-dependent deformations of weak rocks are due to squeezing behaviour of the rock mass, caused by overstress conditions around the tunnel. Squeezing can occur both in weak rock formations (such as schists, claystones, shales, etc.) and in highly jointed rock masses and it is characterised by yielding under the redistribution state of stress during and after excavation [10] that results in the reduction of the tunnel cross-section. The magnitude of the ground movements (tunnel convergence) depends on the type of rock, rock mass strength and deformation properties, and in situ stress conditions [3]. Squeezing is enhanced by unfavourable orientation (parallel to the tunnel axis) of discontinuities (bedding planes, schistosity), and it is found to occur frequently in contact and tectonised zones and faults [6].

As reported by Barla [15] several empirical, e.g. [16, 17], and semi-empirical, e.g. [18-20], approaches are available in literature to predict squeezing. All these methods use as starting point the ratio of uniaxial compressive strength σci/σcm of intact rock/rock mass to overburden stress $p_0=\gamma H$, where γ is the rock mass unit weight and H is the overburden depth [6].

According to Loew et al. [6], the most used approach is the one proposed by Hoek [21]. By means of axisymmetric finite element analyses, he provided an approximate relationship to calculate the tunnel strain ε_t (defined as ratio of radial tunnel displacement to tunnel radius) as a function of the ratio σ_{cm}/p_0 and the internal pressure p_i:

$$\varepsilon_t(\%) = 0.15\left(1-\frac{p_i}{p_0}\right)\frac{\sigma_{cm}}{p_0}^{-\frac{\left(\frac{3p_i}{p_0}+1\right)}{\left(\frac{3.8p_i}{p_0}+0.54\right)}} \tag{1.1}$$

The curve defined by Eq. 1.1 (for zero support pressure pi) may give a first estimate of tunnel squeezing problems associated with different tunnel strain levels (Figure 1.1).

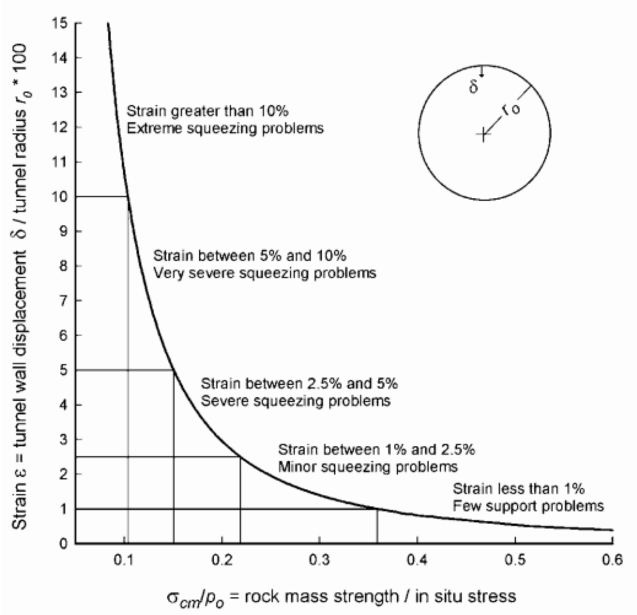


Fig. 1.1 *Squeezing associated with different levels of strain [21].*

The strongly squeezing ground is one of the most common hazards connected to a fault zone in deep Alpine tunnelling, together with high water inflows and structurally controlled failures (Figure 1.2).

Fig. 1.2 *Examples of structurally controlled instability in fault zones: a) highly anisotropic failure at the tunnel sidewalls due to the presence of well-developed foliation or sub-parallel fractures with small spacing; b) progressive block destabilization at the tunnel walls/crown; c) complete collapse resulting from total loss of confinement. After [6].*

In all the described instability phenomena, performance of the TBMs could be significantly reduced.

One of the main drawbacks of TBM tunnelling in faulted/highly fractured zones is a significant reduction of the advancement rate because of large/very large construction delays which are generally associated to the following problems:

- the need to install heavier supports in very fractured/bulking rocks;
- the need to stop the TBM if ground treatments are required;
- the need to install additional drainage systems and waterproofing measurements;
- the need to free the machine if jamming of TBM cutterhead and shield occurs in case of very high tunnel displacements (fact which also poses serious issues of safety for the tunnel crew).

Figure 1.3 illustrates schematically the tunnelling and stability problems that could be encountered when a TBM crosses fault rocks [8].

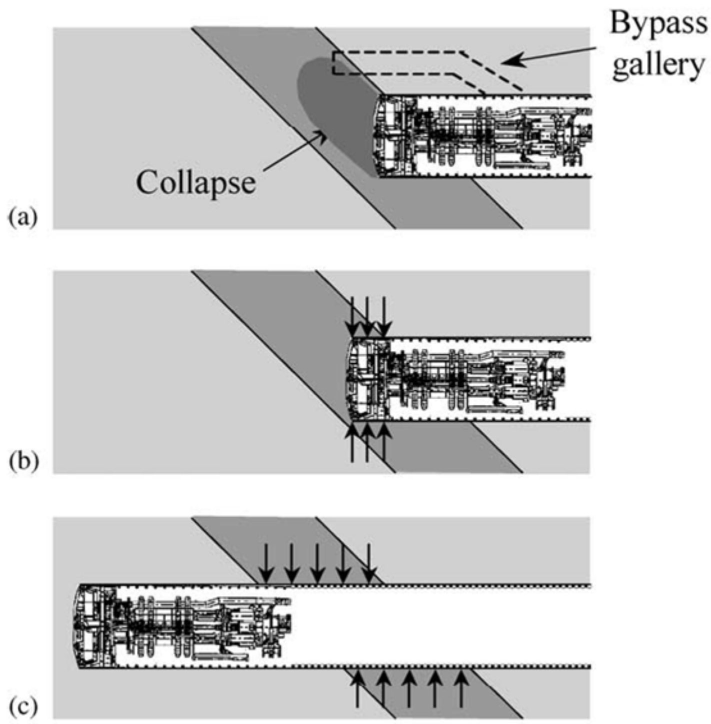


Fig. 1.3 *Progress difficulties of a TBM in tectonically crushed rock zone: a) excavation face instability; b) decompression and blocking of the TBM; c) creep phenomenon [8].*

Furthermore, the presence of a faulted/heavily jointed tunnel face may strongly affect the interaction between rock mass and TBM cutters (the excavation may not proceed anymore via the usual chipping process).

In order to update continuously the ground model, a constant observation of rock mass behaviour during tunnel construction is necessary, evaluating all recorded data. Besides an adequate ground investigation activity (both prior and during construction), that should include direct methods and geophysical methods, the prediction of the TBM performance in such difficult conditions becomes extremely important for planning purposes and for the selection of the proper excavation method.

To evaluate the immediate response of weak rocks to tunnelling, Klein [3] developed the flow chart reported in Figure 1.4. He defined the mass behaviour as “a function of the stresses imposed on the rock surrounding the tunnel and the strength of the rock”. In order to assess the potential for overstressed conditions the modified overload factor (OFM), developed by Deere et al. [22], has been used:

$$OFM = \sigma_\theta / UCS \qquad (1.2)$$

In Eq. 1.2, σ_ϑ is the tangential stress and the UCS the uniaxial compressive strength of the rock. The overstressed rock conditions develop around the tunnel if OFM is greater than one; otherwise potentially firm ground conditions are expected. According to Klein [3], depending on the stress-strain characteristics of the rock, the behaviour under overstressed conditions changes: rocks that fail in a brittle manner (typically coarse-grained rocks such as sandstone and conglomerate) will fracture; rocks that exhibit ductile characteristics (typically fine-grained rocks such as shale, marl and weathered/altered rocks) will yield leading to convergence around the tunnel.

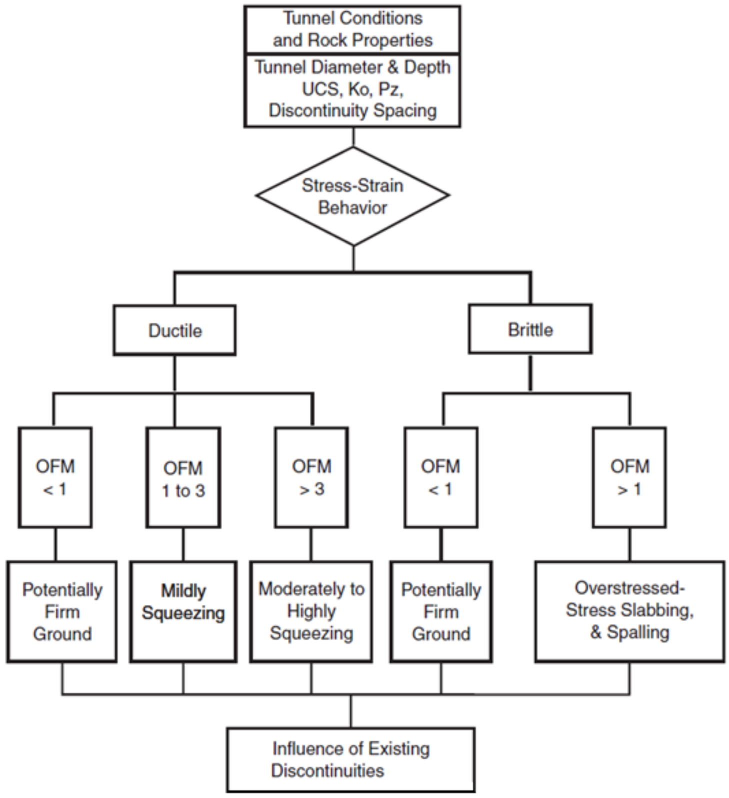


Fig. 1.4 *Immediate response of weak rocks in tunnel [3].*

1.2 Fault zones and TBM

Several cases of TBM tunnelling in fault zones have been recorded in these last decades, in different parts of the world. Some of them, reported in following sections, have been collected by Barton [23] on the basis of his "fault zone experiences" in Italy, Greece, Kashmir and Taiwan.

Switzerland is one of the countries where the underground constructions are considered to be necessary. More than 60% of the area is mountainous and during these last decades several TBM tunnel projects have been carried out or are still on going. The most famous example is the Gotthard Base Tunnel (GBT), at the moment the world's longest railway tunnel.

1.2.1 Pont Ventoux (Italy)

The Italian Pont Ventoux HEP headrace tunnel, excavated by a gripper TBM, was characterised by several fault zones that had disastrous results on the machine progress (Figure 1.5).

The cutter-head was stuck for 6 months, because of the need to probe ahead of the tunnel face, mapping the extent of fracture zones (by means of tomographic images) and then deciding the stabilization measures to be adopted [1]. The results of this condition was an advancement of only 20 m in 7 months. The biggest problem was the combination of adverse water pressures, erosion of faulted material and major void formation due to generally sub-parallel fault orientations. The TBM grippers represented a further complication due to the pressure exerted on the continuous steel liner (flange-to-flange), which was installed in the most adverse rock conditions. This loading caused the crushing of the steel sets with consequent tunnel stability issues.

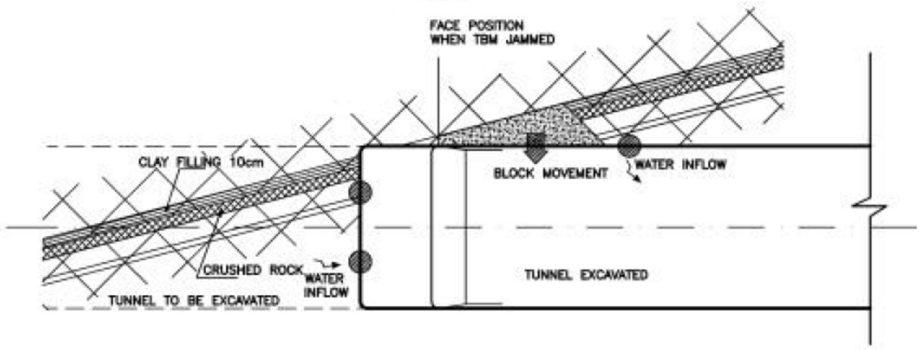


Fig. 1.5 *The TBM jamming as a consequence of overstressing and a 25 cm block movement of the right sidewall [1].*

1.2.2 Evinos-Mornos (Greece)

Another TBM project characterised by adverse geological conditions is the Evinos-Mornos tunnel (Greece), one of the longest hydraulic tunnels in the world.

When the double shield TBM entered a very disturbed zone characterised by a disintegrated rock mass structure, a big collapse occurred in front of the TBM cutter-head (Figure 1.6). The result was the creation of a large cavity, more than 10 m height, above the machine area. Hand-mining operations were required to restart the TBM excavation which caused a 50 days standstill [24].

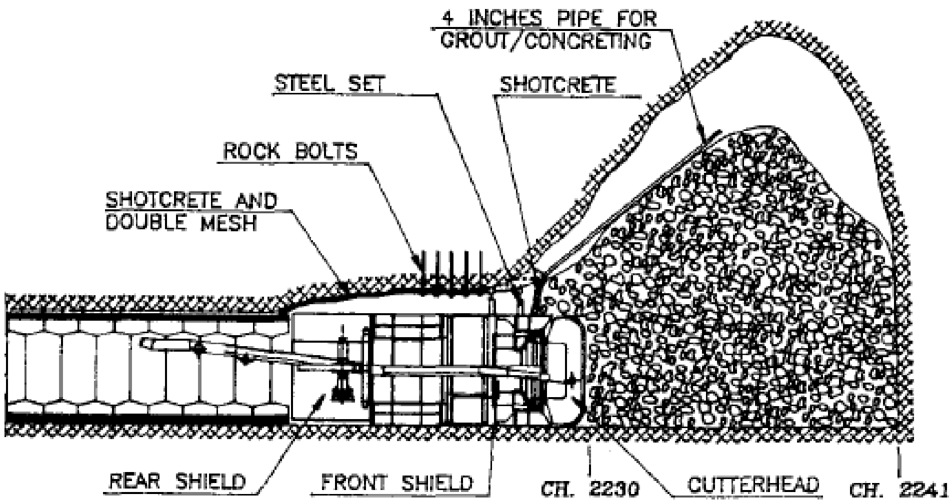


Fig. 1.6 *The big collapse in front of the cutter-head that creates a cavern of more than 10 m high [24].*

1.2.3 Dul Hasti (Kashmir)

The excavation of Dul Hasti Hydroelectric Project (Kashmir, India) was partly performed by an open TBM and it was characterised by serious difficulties.

The machine remained stuck several times in different geological accidents, due to extreme pebble/sand blow-out and stand-up time problems, accompanied by water discharge up to more than 1000 lit/sec (Figure 1.7). Average monthly rates went down to 22 meters per month [25]. Although additional supports, grouting of the cavities in the machine area, drilling of drainage holes and a final modification of tunnel alignment were performed, the heavily damaged TBM was abandoned after a further blockage, and the excavation was resumed with the Drill-and-Blast method.

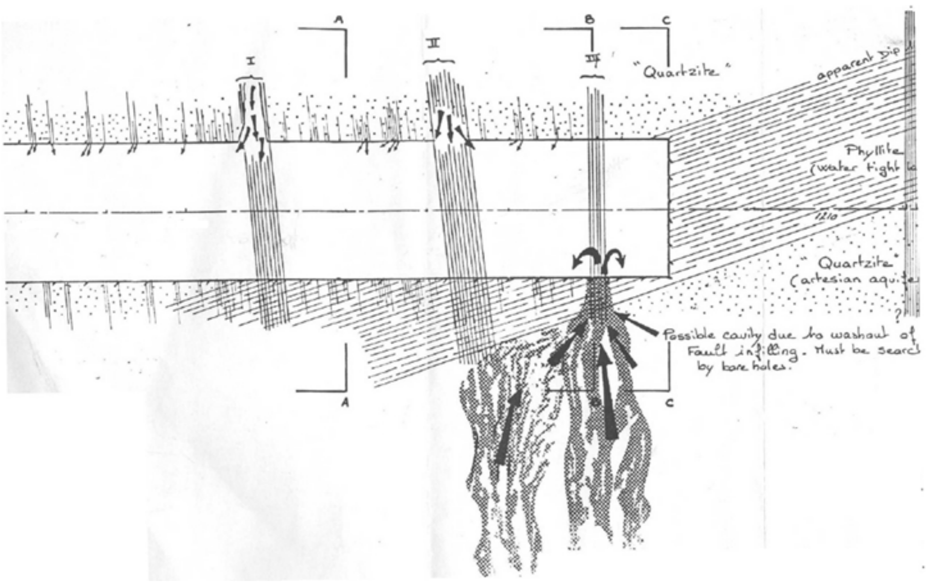


Fig. 1.7 *Cavity formation in the right part of the invert as a consequence of a very important water inflow carrying silt, sand and debris [25].*

1.2.4 Pinglin (Taiwan)

The Pinglin Highway tunnel in Taiwan is an example of particularly complex geological conditions such as sudden inrush of great quantities of groundwater through zones of faulted and fractured rocks. As illustrated in Figure 1.8, this phenomenon contributed to several face collapses and TBM jamming [1].

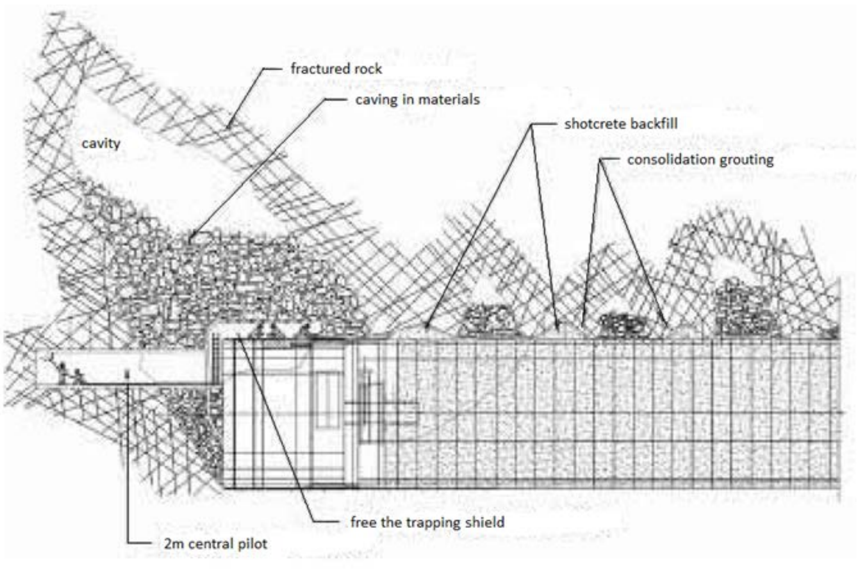


Fig. 1.8 *Example of instability problem at the tunnel face [1].*

Serious faults caused such large cumulative delays, that Drill-and-Blast method was required to complete the 15 km long tunnel, after some 12 years.

1.2.5 Gotthard Base Tunnel (Switzerland)

The Gotthard Base Tunnel (GBT) is a Swiss railway tunnel expected to open in 2016. It is part of the AlpTransit project, also known as the New Railway Link through the Alps (NRLA).

Despite most of problematic zones have been identified in advance along the alignment, some relevant unexpected events occurred during the tunnel excavation. In the south part of the GBT, after about 200 meters, the TBM bumped into a sub-horizontal fault zone characterised by kakirite, cataclasite and highly fractured rocks. It was a few meters thick zone and it affected the west tube for about 100 meters and the east tube for 400 meters, especially at the crown. The detachment of loose material, above and just behind the cutterhead, caused the formation of cavities till 6 m high, which were filled with shotcrete and, in some cases, with in-situ casted concrete. The support was immediately installed, adopting steel sets, welded wire mesh and shotcrete. Because of the difficult ground conditions, the TBM advancement could not exceed 2.5 meters per day. Toward the end of the same excavation section, two nearly parallel sub-vertical fault zones were encountered (Figure 1.9).

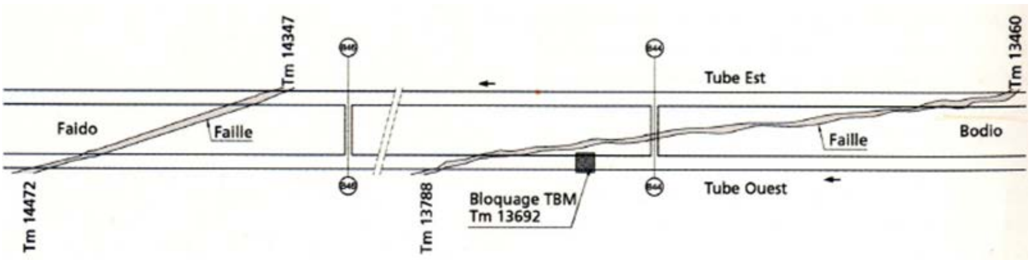


Fig. 1.9 *Location and orientation of the two sub-vertical fault zones [26].*

They were characterised by cataclastic rocks and kakirite layers, the thickness was about 3-5 meters and highly fractured rocks separated them. When TBM went closer these zones, the great pressure exercised on the cutterhead shield led to the machine jamming, with a downtime of ten days. It resumed to bore after the excavation of a cavity at the crown. The convergence of the tunnel caused also important deformations and damages to the supports, for this reason reprofiling works became necessary. Without considering the stoppage of the TBM, the advancement was between 7 and 15 meters per day. The performance proved to be better than the one recorded at the previously described fault zone probably thanks to the experience gained by the crew and to the improvements in installing supports [26].

1.2.6 Moutier Tunnel (Switzerland)

The Moutier Tunnel is situated on the Motorway A16 (Transjurane) in the Canton of Bern (Switzerland).

The excavation was particularly difficult because of several perturbed geological areas, in some cases water bearing, generally constituted of strongly tectonised and partially weathered molasses layers [27]. One of the disturbance zones was encountered after less than 200 m of TBM excavation causing a cave-in at face and subsidence at surface. The shielded TBM was blocked for several months after that the immediate measures adopted for consolidating the ground proved to be useless for the restarting of the machine. In order to continue the excavation, the conventional method was adopted because of the risks related to the TBM as further faults and perturbed zones were expected. A roadheader was used for the remaining of the TBM tunnel heading, along with pipe screen support and steel arches. Once the TBM was freed (Figure 1.10), it was used in order to excavate only the bench and the invert arch as well as the installation of the lining segments up to the northern portal where the machine was dismantled [27].

Fig. 1.10 *The cutter-head was exposed after more than two years (http://www.a16.ch).*

1.3 TBM performance prediction

In order to reduce the machine delays as much as possible, the selection (or designing) of the most suitable TBM has become extremely important, as well as the prediction of the machine performance. During the last decades a great amount of research has principally focused on the prediction of TBM performance in very hard rocks, where the main issues are the rock boreability and the abnormal cutters and buckets wear. Because of the paramount importance of the TBM progress for the time scheduling of a tunnelling project, in these last decades many TBM performance prediction models have been developed following different approaches.

The empirical models represent the largest group. They are based on the recorded machine performances of past tunnelling experiences. They result from regression analyses between rock mass parameters (rock strength, rock fracturing, joint orientation, etc.) and machine parameters such as rate of penetration, cutter force, advance rate, etc. Among them, the Norwegian Institute of Technology model (NTH) is certainly the most widely used. The model estimates the advance rate for a given level of machine thrust adopting standard utilisation factors. The performance is achieved in the field as a whole system, incorporating the effects of discontinuities and rock mass.

On the other hand, the so-called theoretical models are more flexible with machine design parameters than empirical methods. They analyse the forces acting on the cutters or the specific energy required to excavate a unit volume of rock, in order to relate them to the intact rock and rock mass properties and to the machine specifications. The most famous is the one by the Colorado School of Mines (CSM). This model estimates the advance rate from a relationship among thrust, torque and penetration for a certain machine, and allows for an improvement of the TBM cutterhead design.

Furthermore, expanded rock mass classification systems, such as QTBM and RME, have been specifically developed for predictive purposes and for the selection of the most suitable TBM type when planning tunnel construction. Other studies have been performed in order to estimate the boreability of the rock mass with empirical equations based on relationships among intact rock and rock mass parameters like strength, brittleness, RQD, joint spacing and orientation.

Difficult ground conditions, like faulted and highly fractured rocks, have always represented an enormous limit in predicting TBM behaviour, because of a lack of warning during excavation or an underestimation of the problem not well understood ahead of time. Many aspects must be taken into account and several events can take place at the same time such as major high pressure inflows, outwash of fines, large tunnel deformations and formation of cavities at the crown or ahead of TBM, making the situation dramatic and very problematic to deal with.

In the following sections, a brief description of the most widely recognised performance prediction models is reported, trying to underline the limitations of their application in fault zones and/or highly fractured rocks.

1.3.1 The NTH model

The NTH prediction model [28] is based primarily on empirical correlation between geological/rock mechanical parameters and actual tunnelling performance (Table 1.1). Time and cost curves for the various tunnelling operations have been established by collecting and analysing a large amount of information on machine performance and rock mass properties from tunnels in Europe. The model has been continuously upgraded and improved as tunnelling and TBM data from new projects became available. The 1998 version [29] is based on data from about 230 km of bored tunnels.

The main advantages of the NTH model are the generally comprehensive empirical database where the influence of rock jointing can be taken into account, and, being an empirical model, the natural incorporation of the effects of the ground and the excavation system as a whole in their entire complexity [30]. To take into account the influence of joints, NTH has derived a set of relationships between TBM penetration rate and the fracturing factor, depending on discontinuity spacing and angle with the tunnel axis. The main disadvantage of the model is the predicting ability in new systems that differ from the old ones included in the database.

***Table 1.1** The NTH prediction model: main characteristics, after [31]*

Prediction value	Reference	Rock mass factors	Machine factors
Penetration rate (m/hr) Advance Rate (m/hr)	NTH [29]	Uniaxial compressive strength (UCS) Drilling rate index (DRI) Number of joint sets Joint frequency and joint orientation relative to the tunnel axis Porosity	Cutter force RPM Cutter spacing Cutter size and shape Installed cutterhead power

1.3.2 The CSM model

The CSM model [32, 33] is characterised by theoretical basis, but incorporates some empirical relations as well. It starts from individual cutter forces (normal and rolling) to determine the overall thrust, torque and power requirement on the entire cutterhead (Table 1.2).

***Table 1.2** The CSM prediction model: main characteristics, after [31]*

Prediction value	Reference	Rock mass factors	Machine factors
Penetration rate (m/hr) Advance Rate (m/hr) Some TBM parameters	CSM model [32]	Uniaxial compressive strength (UCS) Tensile strength	Cutter spacing Cutter tip width Cutter radius Cutter force TBM diameter RPM

In order to predict the maximum achievable penetration, the estimated values are then compared to the installed machine specifications. Thrust normal force, rolling force and specific energy value can be measured by a linear cutting machine (LCM) while a real-life cutter acts on a large rock sample, reducing the scaling effect. The results of the tests should help to calculate TBM performance parameters (such as torque and thrust requirements) which can be used for designing purposes or can be compared with the field performance data recorded by the TBM during the digging.

The CSM model predicts the penetration rate without any consideration given to the influence of existing joints in the rock. To account for these effects, the model makes use of the correlation factors developed for joint effects by the Norwegian Technical University (NTH) mentioned in the previous section.

1.3.3 The Q_{TBM} model

Barton [34] expanded Q-system of rock mass classification to a new Q_{TBM} system for predicting penetration and advance rates of the TBM, accounting for average cutter force, abrasive nature of the rock, rock stress level and orientation of joint structures (Table 1.3). The new parameter Q_{TBM} is a function of several basic parameters, many of which can be simply estimated by an experienced engineering geologist [35].

The development of a relationship between penetration rate and Q_{TBM} was based on a process of trial and error using case records. To estimate the advance rate of the TBM, Barton adopted the function Tm as utilisation factor (which expresses the fraction of time in which the machine has been effectively used to bore the rock), where T is the time in hours and m is a negative gradient that express the decelerating average advance rate as the unit of time increase. Although the proposed relationships are very simple to use, the great number of parameters involved gives this model limited use in practice.

***Table 1.3** The QTBM prediction model: main characteristics, after [31]*

Prediction value	Reference	Rock mass factors	Machine factors
Penetration rate (m/hr) Advance Rate (m/hr)	Q_{TBM} [34]	RQD_0 J_n, J_r, J_a J_w SRF Rock mass strength Cutter life index (CLI) Quartz content Induced biaxial stress at the face Porosity	Cutter force TBM diameter

1.3.4 The RME model

The RME indicator [36, 37] estimates the performance of double-shield and open-type TBMs on the basis of statistical analyses of data from more than 500 tunnel case records. The RME is calculated using five input parameters specifically related to rock mass behaviour and TBM characteristics: abrasion, discontinuity spacing and orientation, stand-up time, uniaxial compressive strength of intact rock and water inflow at tunnel front (Table 1.4). A certain rating is associated with each parameter, and the sum of them gives the value of the RME index.

***Table 1.4** The RME prediction model: main characteristics, after [31]*

Prediction value	Reference	Rock mass factors	Machine factors
Penetration rate (m/hr) Advance Rate (m/hr) Specific energy (kJ/m^3)	RME [36]	Uniaxial compressive strength (UCS) Abrasion Rock mass jointing at the face Stand-up time Water flows	Total cutterhead thrust RPM Torque

Significant correlations between RME and theoretical Average rate of Advance (ARAT) of the machine have been established to predict the performance of open-type, single-shield and double-shield TBMs. For obtaining the ARAT, the section must meet some characteristics such as a length greater than 30 meters, no significant variations in RME and RMR values and a TBM utilisation factor between 30% and 60%. The latter could represent a limit for the application of this indicator in fault zones, where the utilisation factor reaches values much lower than those required to apply the method.

1.3.5 Other studies on the TBM performance

By considering more recent studies present in literature, two further TBM performance prediction models are given below. In Table 1.5 the main characteristic of these models are reported.

Gong and Zhao [38] proposed a statistical model based on combination between a rock mass conceptual model, for evaluating rock mass boreability, with an established database developed during the construction of the DTSS project in Singapore. On the basis of rock fragmentation mechanism, the conceptual model identifies the influence of rock mass properties on TBM penetration rate, showing that the rock uniaxial compressive strength and the volumetric joint count seem to have the most relevant effect. However, the model presents some limitations and attention must be paid when it is used to predict TBM performance with critical different tunnelling conditions.

Another new prediction method, developed by Hassanpour et al. [39], is proposed to evaluate rock mass boreability. This empirical model is based on the analysis of data obtained from four tunnelling projects, with a total length of 55 km which have been excavated in different geological conditions. Relationships between geological and TBM operational characteristics are investigated, showing strong links between rock mass parameters such as uniaxial compressive strength, RQD and joint spacing, with the so-called Field Penetration Index (FPI) [40]. Although the model is applicable in a wide range of geological conditions (like layered and jointed sedimentary rocks or blocky rocks), extreme caution in highly fractured rock masses, fault zones and water sensitive rocks, is required.

Table 1.5 *Two recent studies on TBM-performance prediction: main characteristics , after [31]*

Prediction value	Reference	Rock mass factors	Machine factors
Boreability Index, BI (kN/mm/rev)	[38]	Compressive strength Volumetric joint count Brittleness index Angle between main discontinuities and tunnel axis	Cutter force
Field Penetration Index, FPI (kN/mm/rev)	[41]	Uniaxial compressive strength and RQD	Cutter force RPM

As previously said, fault zones or highly fractured rocks, represent an enormous limit in predicting TBM behaviour. As a matter of fact, since many events can take place at the same time (e.g. major high pressure inflows, outwash of fines, large tunnel deformations and formation of cavities at the tunnel crown or ahead of TBM) making in some cases the situation dramatic and very difficult to deal with, several parameters must be taken into account.

Sapigni et al. [35] tried to find possible relationship between the machine performance and the geological/geotechnical characteristics of the rock mass. According to these results the empirical correlation between the rate of penetration of the machine and the Rock Mass Rating (RMR) seems to follow a bell-shaped curve (Figure 1.11), with the maximum TBM performance reached for rock masses of medium quality (i.e. RMR = 40 ÷ 60). At the same time, the penetration rate of the TBM decreases for very poor or very hard rock masses.

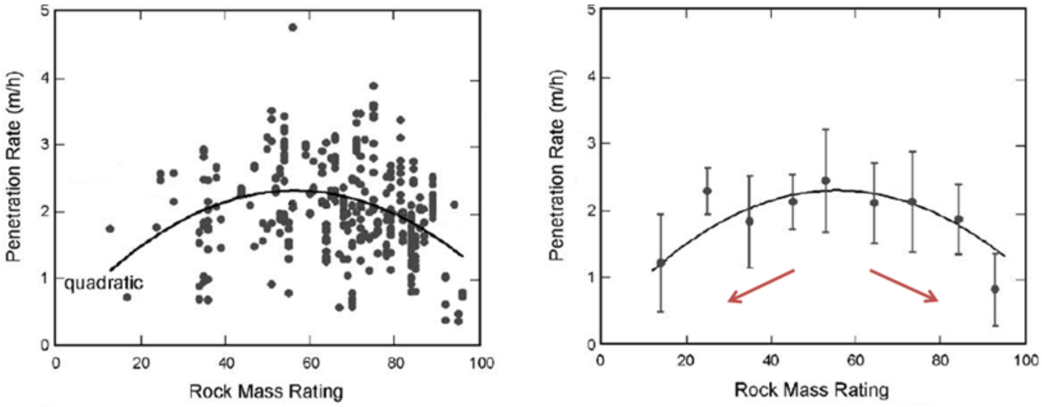


Fig. 1.11 *An example of correlation between the rate of penetration of the TBM and Rock Mass Rating - raw and average data, after [35].*

Bad boreability in adverse ground conditions could be a consequence of mucking problems and face instabilities that limit the potentially high penetration rate as a consequence of thrust reduction [35].

In literature it is also possible to find examples of rating system formulations used for estimating the utilisation factor of the TBM [42, 43], where an important role is played by the contractor experience (especially in the thrust force control).

However, as reported by Einstein [44], it is practically impossible to predict exactly the TBM performance. As a matter of fact, the definition of the utilisation factor is based on a combination of parameters which are not only related to the rock mass conditions. A very important role is indeed played by non-quantifiable factors such as the skills of the tunnel crew, the variable performance of the equipment, as well as accidents/breakages occurring at various intervals during construction.

1.4 Most common rock mass parameters in TBM-performance prediction

A statistical analysis on several prediction models has been carried out with the aim of valuing in which frequency the different geological-geotechnical characteristics are involved. In Figure 1.12 the results of this investigation is reported.

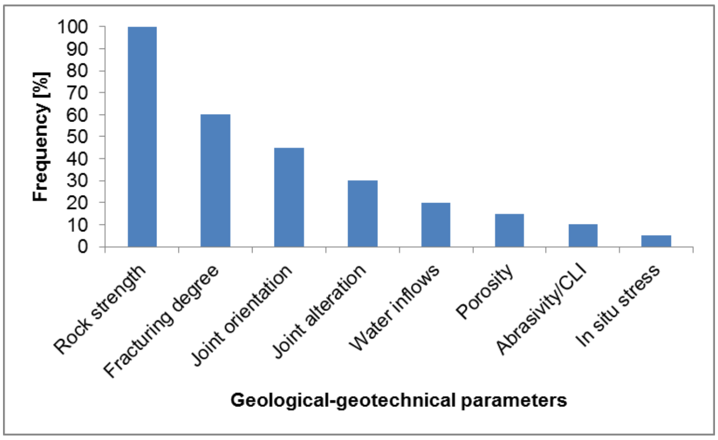


Fig. 1.12 *Geological-geotechnical parameters generally evaluated for TBM-performance prediction. The frequency refers to a sample of 20 existing prediction models.*

The most common characteristics are the strength of the rock (in the 100% of the analysed models) and the degree of fracturing (in the 60% of the analysed models). The "rock strength" term includes closely related parameters such as the uniaxial compressive strength, the brittleness as well as the hardness of rock. Also the fracturing degree refers to several factors (e.g. joint spacing, RQD and J_v) that have been grouped because representative of the same information. The other parameters highlighted by this analysis (joint orientation and alteration, possible water inflows, porosity of the rock, abrasion/quartz content as well as the cutter life index and the in situ stress) come out less used in order to estimate the performance of the TBM. However the characteristics of joints, in terms of strike and weathering conditions, make a considerable impact on performance prediction being taken into account in the 30-40% of the analysed models.

Based on the previous considerations, in order to carry out the preliminary analyses reported in the following sections, the uniaxial compressive strength (UCS) and the volumetric joint count (J_v) will be selected as representative parameters for the material strength and for the fracturing degree of the rock mass.

2 Fault zone: formation and structure

“Fault zone” is the term commonly used to identify a brittle shear zone generated in the upper part of the Earth’s crust (Figure 2.1) and it refers to one of the shear zone categories which are defined based on the dominant type of rock deformation.

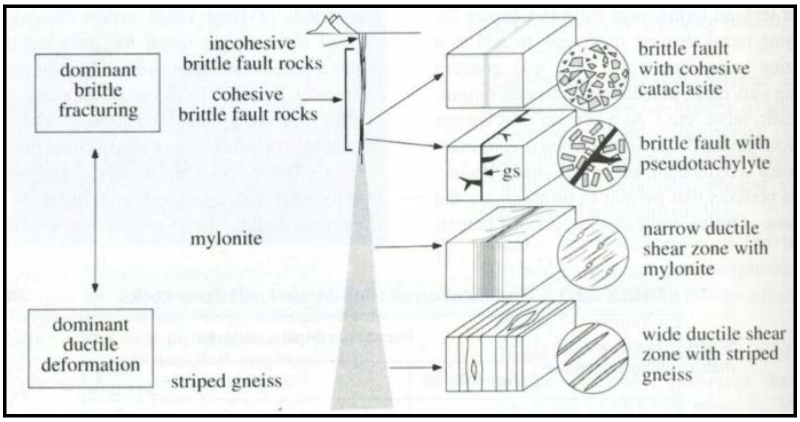


Fig. 2.1 *Conceptual model showing the distribution of shear zones with depth [45].*

The shear zones are regions, usually much longer than thicker, composed of highly strained rocks. There is a spatial gradient in the amount of strain that is generally highest within the centre of the shear zone, decreasing outward into its adjacent rocks [6].

The response of the rock to the rate of shear which it is subjected to, depends on the pressure and temperature conditions and determines how the rock accommodates the deformation. The mechanism of shearing can be occurred at brittle, ductile or intermediate conditions, identifying four mainly types of shear zones [6, 46-48]:

- the brittle shear zones (fault zones), that contain products of brittle deformation mechanisms, considering the potential subsequent weathering or cementation by migrating fluid [49] promoted by the high permeability of the fractured material;
- the ductile shear zone, where the deformation processes are mainly achieved by crystal plasticity, involving a minor amount of fracturing;
- the semi-brittle shear zones, that involves mechanisms such as cataclastic flow and pressure solution;
- the brittle-ductile shear zones, that shows evidence of both brittle and ductile deformation.

The ductile (and semi-brittle) shear zones do not generate instability problems in underground construction [6], while the most problematic events related to rock engineering, are due to brittle shear zones (fault zones).

Rocks produced by brittle faulting, can show properties of hard rocks or else of soil-like material depending on the cohesive or non-cohesive character, in relation to the variable ratio of gouge matrix to rock blocks with different sizes, shapes and strengths (Medley [50], defined as “bimrocks” the block-in-matrix rocks). Therefore, the weak rocks inherited from brittle processes are characterised by a low mechanical strength situated somewhere between that of rock and soil. They additionally show a high deformability involving non-linear constitutive laws, a strong strength dependence on water saturation and temperature, and a great susceptibility to weathering.

Fault rocks (generally termed cataclastic rocks) result from a mechanical and chemical weathering related to tectonic activity [51]. This degradation process is known as cataclasis, a friction-dependent mechanism of deformation that involves brittle fracturing, grinding, crushing and rotation of rock components. Mechanical processes are characterised by stress and strain localisation that is responsible for fracturing at different scales. Chemical processes, that transform the primary rock composition and lead to the development of secondary mineral phases (phyllosilicates, clay minerals), increase in case of high degree of fracturing which favours fluid circulation.

Cataclasis occurs in a regime where continuous fracturing becomes the energetically most favourable deformation mode, while movements along pre-existing discontinuities, governed by a frictional sliding criterion, are made impossible [51]. From Figure 2.2, it is possible to observe that at the same confining pressure, frictional sliding requires a greater differential stress than fracturing during a cataclastic flow. The Mohr circle I indicates critical stress conditions for shear fracture, the Mohr circle II frictional sliding on a fracture plane at the same confining pressure and the Mohr circle III the critical stress conditions for cataclasis process.

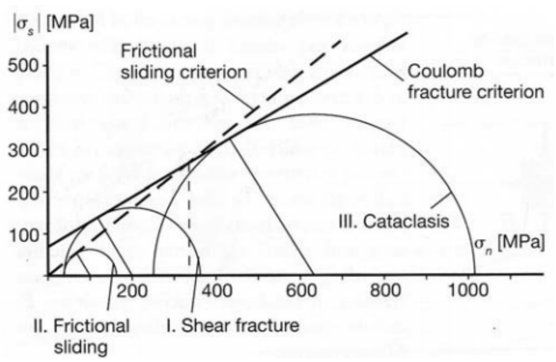


Fig. 2.2 *Representation of Coulomb fracture criterion and frictional sliding criterion for sliding on a pre-existing fracture on a normal vs. tangential stress diagram [51].*

Cataclastic fault zones are structurally characterised by two different areas showing contrasting rheological relationships: the fault core zone and the damage zone [51].

The fault core is a "structural, lithologic and morphologic portion of a fault zone where most of the displacement is accommodated" [52]. It consists of low permeability cataclastic rocks, characterised by a high amount of fine-grained matrix in comparison to clasts [51].

The damage zone bound the fault core and it is characterised by subsidiary structures such as small faults, veins, fractures and fold, causing heterogeneity and anisotropy in fault zone permeability that is controlled by the hydraulic properties of the fracture network [52, 53]. A conceptual model of the fault zone architecture is illustrated in Figure 2.3.

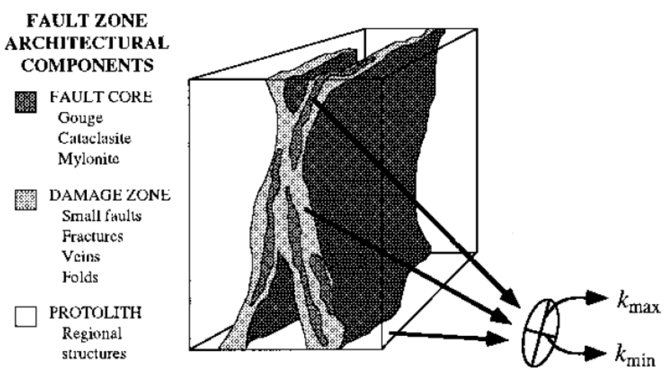


Fig. 2.3 *Conceptual model of fault zone without undeformed protolith. The "ellipse represents magnitude and orientation of the bulk two-dimensional permeability (k) tensor that might be associated with each distinct architectural component of fault zone" [52, 54, 55].*

A relatively undeformed protolith surrounds the fault core and damages zone. This country rock reflects the fluid flow and elastic properties of unfaulted host rock [52].

In Figure 2.4 possible permeability structures of a fault zone are shown: the fluid flow within and near the fault zone is controlled by the distribution of each component [52].

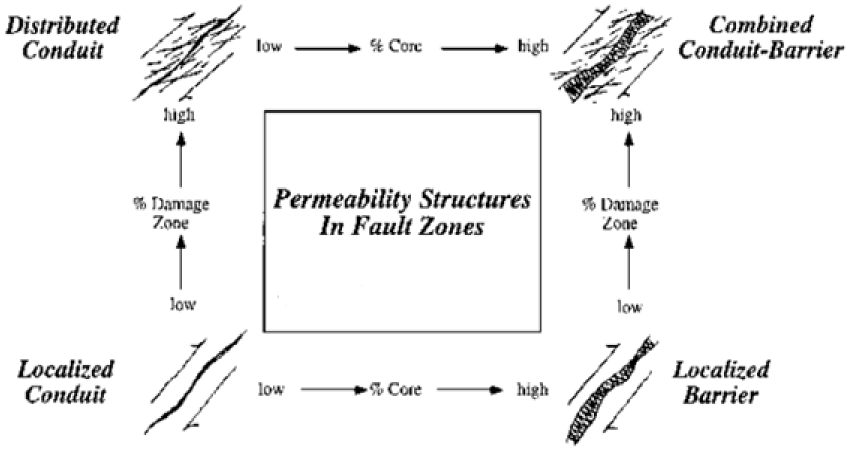


Fig. 2.4 *Conceptual scheme for fault-related fluid flow. Permeability reduction due to grain-size reduction and/or mineral precipitation that fill open pore spaces, leads to fault cores that act as barriers to fluid flow [52].*

2.1 Terminology and geological classifications of fault rocks

Several geological classifications of fault rocks have been proposed in literature (e.g. [46, 49, 56-59]). They are generally based on parameters such as fabric, texture and clasts size or they take into account the cohesive/non-cohesive character of materials (Table 2.1).

According to the conventional engineering geological terminology, cataclastic rocks can be divided in the following classes:

- fault breccia, it consists of large fragments of rock in a fine-grained matrix that could include mineral veins formed in voids between the clasts. It may have a non-cohesion (with visible fragments >30%) or a cohesion character. In the cohesive (and cemented) fault breccia the proportion of matrix is less than 10% and, based on the fragments dimension, it is possible to define crush breccia (>0,5 cm), fine crush breccia (>0,1 cm and <0,5 cm), crush microbreccia (<0,1 cm);
- fault gouge, a non-cohesive clay-rich and fine-to-ultrafine-grained material (with visible fragments <30%) that may possess a schistosity;
- cataclasite, normally cohesive and non-foliated, consisting of angular clasts in finer-grained matrix. It may be subdivided according to the relative proportion of finer-grained matrix into protocataclasite (10-50%), mesocataclasite (50-90%) and ultracataclasite (90-100%);
- pseudotachylite, a cohesive and ultrafine-grained rock, it is a vitreous-looking material and dark in colour. It is generally found along a fault surface or as thin planar veins injected into the fractures in the host rock.

The relationship of other types of rocks to the cataclastic ones has been the subject of a considerable confusion for many years, especially with regard to mylonite.

Mylonite is a fine-grained cohesive rock with at least microscopic foliation [58]. It was originally defined as a cataclastic rock but it is now clear that the cataclasis is not the dominant process leading to the formation of mylonite series textures [46]. Nowadays it is seen as the product of ductile processes occurring at depth, below the brittle shear zone (as shown previously in Figure 2.1), under such heat and pressure that the mineral deform in a plastic way (ductile shear zone).

Table 2.1 *Textural classification of fault rocks, after [46]*

<table>
<tr><td></td><td colspan="3">RANDOM-FABRIC</td><td colspan="5">FOLIATED</td></tr>
<tr><td rowspan="2">INCOHESIVE</td><td colspan="3">FAULT BRECCIA
(visible fragments > 30% of rock mass)</td><td colspan="5">?</td></tr>
<tr><td colspan="3">FAULT GOUGE
(visible fragments < 30% of rock mass)</td><td colspan="5">?</td></tr>
<tr><td rowspan="6">COHESIVE</td><td>Glass/devitrified glass</td><td colspan="2">PSEUDOTACHYLYTE</td><td colspan="5">?</td></tr>
<tr><td rowspan="4">NATURE OF MATRIX
Tectonic reduction in grain size dominates grain growth by recrystallization and neomineralization</td><td colspan="5">CRUSH BRECCIA (fragments >0.5 cm)
FINE CRUSH BRECCIA (0.1 cm< frag.s <0.5 cm)
FINE MICROBRECCIA (fragments <0.1 cm)</td><td>0 - 10 %</td><td rowspan="4">PROPORTION OF MATRIX</td></tr>
<tr><td>PROTOCATACLASITE</td><td rowspan="3">Cataclasite Series</td><td colspan="2">PROTOMYLONITE</td><td rowspan="3">Mylonite Series</td><td>10 – 50 %</td></tr>
<tr><td>CATACLASITE</td><td rowspan="2">PHYLLONITE VARIETIES</td><td>MYLONITE</td><td>50 – 90 %</td></tr>
<tr><td>ULTRACATACLASITE</td><td>ULTRAMYLONITE</td><td>90 – 100 %</td></tr>
<tr><td>Grain growth pronounced</td><td colspan="2">?</td><td colspan="5">BLASTOMYLONITE</td></tr>
</table>

The texture of mylonite largely depends on the interplay of strain and recovery rates [58]. Through the competition between strain and recovery/recrystallization mechanism in the matrix, Wise et al. [58] proposed a practical framework for terminology of fault-related rocks (Figure 2.5). At one extreme, for a modest recovery, there are cataclastic rocks, at the other extreme, the dominant recovery/recrystallization processes yield ordinary metamorphic rocks.

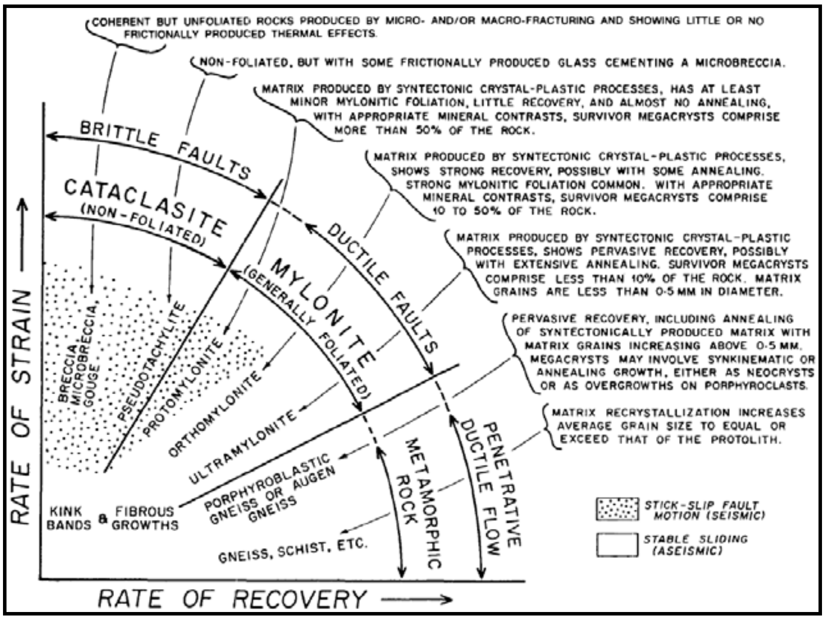


Fig. 2.5 *Terminology of fault rocks depending on the rate of strain versus the rate of recovery [58].*

2.2 Geomechanical characteristics of fault material: weak and weathered rocks

The geomechanical and hydrological properties of fault rocks can be defined by rheology of host rocks, petrography and age of fault activity [60]:

- brittlely deformed incompetent host rocks, such as phyllite, marl and shale, form zones (also known as cohesive kakirites or fault gouge) with soil-like features, characterised by a low compressive strength, low values of Young's modulus and a high squeezing potential. From the hydrological point of view they are characterised by a low permeability which hinders the flow of cementing solutions. For this reason the older fault rocks do not show a very high grade of cementation and the properties do not change significantly compared to younger fault zones;
- brittlely deformed competent host rocks, such as granite, massive carbonate and quartzite, form nearly non-cohesive zones (cohesionless kakirites), extremely permeable, with a high potential of water ingress. The older fault rocks (cataclasite) show an elevated rate of cementation due to high permeability, and geomechanically, they have a quality similar to that of intact rocks (high compressive strength and high values of Young's modulus).

In engineering geology the most problematic and arduous zones, especially in tunnelling projects, are represented by the friable cohesionless kakirites and by the fault gouge zones (characterised by a high squeezing potential), while cataclasites do not play a significant role [60].

One of the main problems is represented by the geotechnical description of the fault zone. The characterisation proves to be very difficult because of the extreme complex structure and heterogeneous nature of the material which is the result of combined contributions of weak and strong rock components. For handling a range of properties intermediate to soil and hard rock weak and weathered rocks should be considered at the same time (Figure 2.6).

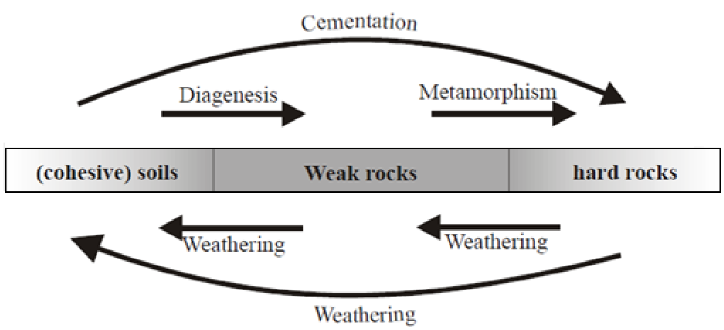


Fig. 2.6 *Position of weak rocks between cohesive soil and hard rocks, after [61].*

Santi and Doyle [62] distinguished weathered rocks from weak rocks, defining them in two different ways:

- weak rocks are intact, unweathered to slightly weathered materials with a low compressive strength, or are materials characterised by a highly fracturing degree;
- weathered rocks have undergone "significant deterioration, particularly near the ground surface or along fractures".

Although these materials show differences in both genesis and post-depositional history, they can be grouped together under the generic term of "weak rock" for analysis and discussion [62, 63].

In Table 2.2 several types of typically weak rocks are grouped by mineral content and post-depositional processes (slaking or dissolution in water, weathering, poor cementation and consolidation, development of planes of weakness) because both of these factors contribute to weak rocks behaviour and formation [63].

Table 2.2 *Types of weak rocks, after [63]*

Mineralogy or processes affecting	Weakness characteristics	Rock types
Materials with high clay content (or other undesirable minerals)	Sensitive to water; poorly cemented; low friction angles.	Overconsolidated clay; cemented clayshale, shale; mudstone, mudrock; tightly interbedded sedimentary rocks; marl; flysch; turbidite; (chalk; anhydrite; gypsum)
Young materials	Poorly cemented; poorly consolidated; significant component of unstable and easily weathered minerals.	Quaternary carbonates; Tertiary sedimentary rocks (conglomerate, sandstone); Tertiary volcanic rocks (some basalt, tuff, volcanic breccia); pediments
Highly weathered materials	Loss of mineral framework and cohesive bonds; creation of weakness planes and clay because of weathering; lower friction angles.	Saprolite; weathered igneous rocks; weathered metamorphic rocks
Metamorphosed materials	Unstable and easily weathered minerals; layered weakness planes or block-in-matrix structure.	Melange; metashale
Hardened soils	Possible collapse upon extended saturation.	Hardpan; caliche; laterite; tropical duracrust

In order to better describe weak and weathered rocks, several classifications have been developed, e.g. [63-67]. Most of these systems were born for granite and some of them have been improved in order to be applicable to a wider range of material types.

Dearman [68] provided two descriptive schemes of weathering grades for rock material (Table 2.3) and for rock mass (Table 2.4), based on several factors such as changes associated with mechanical and chemical weathering, degree of discolouration, rock to soil ratio and presence or absence of the original rock texture.

Table 2.3 *Description of weathering grades of rock material, after [68]*

Term	Description
Fresh	No visible sign of weathering of the rock material.
Discoloured	The colour of the original fresh rock material is changed and is evidence of weathering. The degree of change from the original colour should be indicated. If the colour change is confined to particular mineral constituents this should be mentioned.
Decomposed	The rock is weathered to the condition of a soil in which the original material fabric is still intact, but some or all of the mineral grains are decomposed.
Disintegrated	The rock is weathered to the condition of a soil in which the original material fabric is still intact. The rock is friable, but the mineral grains are not decomposed.

According to Dearman [68, 69], “mechanical weathering results in the opening of discontinuities by rock fracture, the opening of grain boundaries, and the fracture or cleavage of individual mineral grains”, while chemical weathering “leads to the eventual decomposition of silicates to clay minerals”. The discolouration of the rock represents the early stages of chemical weathering.

Santi [63] proposed two field classifications for weak and weathered rocks with the aim to provide more useful systems to be easily applied (without special equipment and laboratory work) and able to suggest at least a range of engineering properties on the basis of a qualitative analysis.

The first one is based on a modification of Dearman's system in order to consider also non-granitic materials such as shales. The weathered materials, such as igneous and metamorphic rocks, indicate quite strong rocks in the fresh state which tends to weather into stronger blocks surrounded by matrix. The weak materials, such as overconsolidated clays, shales and mudstones, refer to moderately weak rocks in the fresh state, and they tend to weather more homogeneously throughout the mass, with a major intensity near the ground surface. The second classification system incorporates the modified Dearman's scheme and other existing systems taking into account several factors such as the influence of the matrix on the overall behaviour, the rock strength and the discontinuities characteristics (spacing, aperture and roughness).

Table 2.4 *Scale of weathering grades of rock mass, after [68]*

Term	Grade	Description
Fresh	I	No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surfaces.
Slightly weathered	II	Discolouration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering.
Moderately weathered	III	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.
Highly weathered	IV	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.
Completely weathered	V	All rock material is decomposed and/or disintegrated to soil. The original mass structure and material fabric are still largely intact.
Residual soil	VI	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

For each weathering grade Santi [63] finally provided quantitative values of some engineering characteristics such as unconfined compressive strength (UCS), drilling RQD and permeability (Table 2.5).

Table 2.5 *Engineering properties for each weathering grade, after [63]*

Engineering property	Fresh I	Slightly Weathered II	Moderately Weathered III	Highly Weathered IV	Completely Weathered V	Residual Soil VI
Point load strength (MPa)	9-18	5-12.5	2-6	0.3-0.9	0.1-0.5	-
Schmidt hammer value (MPa)	59-62	51-56	37-48	12-21	5-20	-
Moisture content (%)	0.06-0.30	0.15-0.29	0.25-0.49	0.37-3.80	7.84-21.00	12.24-22.1
UCS (MPa)	125-260	100-175	60-120	35-55	1-10	<1
Drilling RQD (%)	75, usually 90	75, usually 90	50-75	0-50	0 or does not apply	0 or does not apply
Core recovery (%)	90	90	90	15-70	15 as sand	15 as sand
Permeability	Low to medium	Medium to high	Medium to high	High	Medium	Low

Several other studies have been performed in order to assess the weathering impact on the geotechnical properties of the rock mass.

Thuro and Scholz [67], for instance, analysed data from the Konigshainer Berge Tunnel Project (Germany) where zones of intensive weathering and hydrothermal alteration affected granites. They provided an overview of some rock parameters determined for each weathering grade mainly referring to the ISRM weathering grades.

In order to describe fault rocks as weak rocks, a summary of geological and geotechnical properties is reported in Table 2.6 starting from a technical literature review compiled by Santi [63]. To better characterise altered/cataclastic rocks, only materials with important degrees of weathering are considered on the basis of the previously reported information.

Table 2.6 *Summary of engineering properties of weak rocks, after [63]*

Test or property	Value or range for weak rock	Reference
Compressive strength	1-40 MPa	[63, 67, 70]
Rock quality designation	< 25-75 %	[62, 63]
Ratio between weathered matrix and unweathered blocks	> 75 % matrix	[71]
Natural moisture content	> 1 % for igneous and metamorphic rocks; > 5-15 % for clayey rocks	[62]
Dearman weathering classification	≥ category 4	[64]
Friction angle φ [◦] (Weathered granites)	Highly/completely weathered: 10-30	[67]
	Residual soil: 30-40	
Cohesion c [kPa] (Weathered granites)	Highly/completely weathered: 50-10	[67]
	Residual soil: 15-35	
RMR (Bieniawski, 1976)	< 35-60	[64]
Norwegian Geotechnical Institute “Q” rating	< 2	[64]
Extended GSI	< 30	[72-74]

2.3 Cataclastic fault rocks characterisation: previous research at EPFL

The geological and geomechanical characterisation of cataclastic rocks has already been object of a research project at EPFL which interested two PhD theses [75, 76].

Burgi [75] developed a new quantitative characterisation system where (in a simplified approach) the mineralogical composition and the rock fabric (texture and structure) represent the main controlling factors for the geomechanical behaviour of the cataclastic fault rocks. The mineralogical-structural parameters have been afterwards correlated with the strength parameters studied and tested by Habimana [76].

Habimana [8, 76] carried out at the EPFL’s Rock Mechanics Laboratory (LMR), a laboratory testing programme on cataclastic rocks (also called tectonically crushed rocks) derived from several tunnel projects and characterised by different tectonisation degrees. This laboratory programme included uniaxial and triaxial compressive test, shear test, brazilian test and creep test.

By a qualitative and quantitative interpretation of the test results, an important correlation between the behaviour of cataclastic rocks and some factors such as the anisotropy, the stress level and the tectonisation degree has been observed.

Based on these considerations, a non-linear failure criterion characterising the strength of material has been suggested, along with a stress-strain relationship and a creep law.

The proposed failure criterion (Eq. 2.1) is an extension of the Hoek and Brown failure criterion:

$$\sigma_1 = \sigma_3 + t\sigma_{ci}\left(m_b \frac{\sigma_3}{t\sigma_{ci}} + s\right)^a \tag{2.1}$$

where σ_1 and σ_3 are the maximum and minimum principal effective stresses at failure, σ_{ci} is the uniaxial compressive strength of the intact rock, m_b, s and a are the parameters related to rock mass characteristics and t refers to the tectonisation degree.

Habimana [8, 76] considered the properties of cataclastic rocks varying from the intact rock properties to the soil ones. These two extreme cases could be represented, respectively, by the Hoek and Brown criterion and by the Mohr-Coulomb criterion (Figure 2.7). Between these limit conditions, the material strength progressively decreases as the intensity of the tectonic degradation process increases [8].

The parameter t varies between 1 (for intact rock) and 0 (for soil-like material).

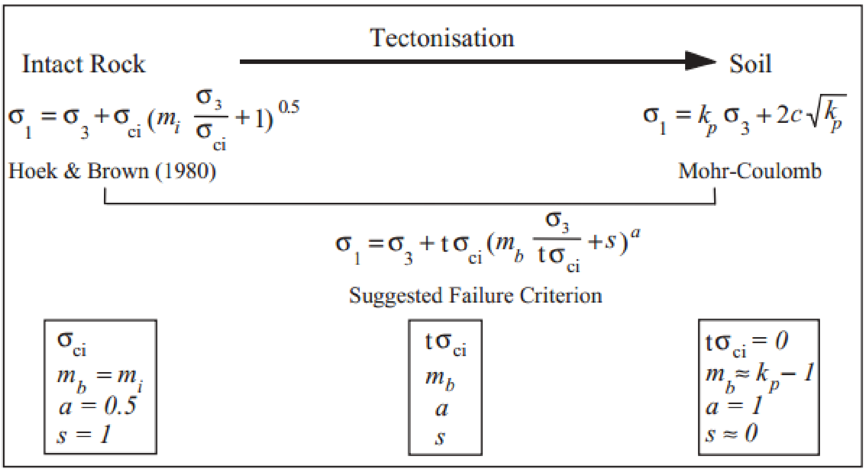


Fig. 2.7 *Progress evolution of a rock mass from intact rock to soil-like material during tectonisation [8].*

In Table 2.7, the relationships proposed by Hoek and Brown [77] between the failure criterion parameters and GSI are reported.

Habimana [8, 76] suggested new correlations (Table 2.8) in order to take into account the weathering degree of the rock mass. Except for the formulation of s that is kept, the original relationships (Table 2.7) proposed by Hoek and Brown [77] for the rock mass parameter a and m_b, have been modified in order to better represent the characteristics of cataclastic and tectonised rocks.

Habimana [8, 76] identified four different state of tectonisation degree: lightly tectonised, tectonised, highly tectonised and extremely tectonised. The last one represents a totally fragmented and disintegrated rock which can be described as a soil-like material.

In order to quantify the tectonisation degree (expressed by the parameter t) the Geological Strength Index (GSI), introduced by Hoek [78] and extended by Hoek et al. [72], has been considered.

Table 2.7 *Relationships between the Hoek and Brown failure criterion parameters and GSI, as proposed by Hoek and Brown [77], after [8]*

GSI > 25	GSI < 25
$s = \exp\left(\frac{GSI - 100}{9}\right)$	$s = 0$
$a = 0.5$	$a = 0.65 - \frac{GSI}{200}$
$m_b = m_i \exp\left(\frac{GSI - 100}{28}\right)$	

Table 2.8 *Relationships between the suggested failure criterion parameters and GSI, after [8]*

Limits	Suggested relationships
$0 \le s \le 1$	$s = \exp\left(\frac{GSI - 100}{9}\right)$
$0.5 \le a \le 1$	$a = \frac{1}{2}\left[1 + \exp\left(\frac{-GSI}{20}\right)\right]$
$(k_p - 1) \le m_b \le m_i$	$m_b = k_p - 1 + (m_i - k_p + 1)\exp\left(\frac{GSI - 100}{28}\right)$
$0 \le t \le 1$	$t = \left(\frac{GSI}{100}\right)^{0.55}$

Based on the laboratory test results obtained by Habimana [8, 76], also the shear characteristics of the filling material (friction angle, φ) seem to play an important and not negligible role in affecting the strength of the cataclastic material. This dependence, which increases with the tectonic degradation degree, is represented by the passive earth coefficient kp defined by the Eq. 2.2:

$$k_p = \tan^2\left(45^\circ + \frac{\varphi}{2}\right) \tag{2.2}$$

The m_b value is therefore bounded between m_i for an intact rock and $(k_p$-1) for a soil-like material.

3 Data collection: development of TBM-performance database

In order to investigate the correlations existing between parameters describing TBM performance and geological/geotechnical characteristics of a highly fractured and faulted rock mass, a TBM-performance database has been developed camping data coming from several tunnel projects.

The attention has been focused on longitudinal tunnel sections in which difficult environments for the TBM operations have been encountered, such as fault zones and highly fractured rock masses. The sections are characterised by homogeneous geological/geotechnical conditions and mostly constant TBM performance over the entire length that varies from 8-10 meters to several tens of meters. A detailed description of the approach adopted to select the tunnel sections is provided in the following sections.

Different tunnel categories have been considered (high speed railway, headrace and tailrace tunnels) and all of them have been excavated by gripper TBMs. An open-type machine is generally used in hard rock and good geological conditions or where considerable deformations are expected. In less favourable ground conditions, such as fault zones and highly fractured rocks, the required additional supports, the potential grouting and significant problems for the crew security make a gripper TBM less efficient compared to a shield-machine. Therefore, through the developed database, it has been possible to analyse the performance of the TBM in such difficult environments, considering the worst excavation conditions represented by the disadvantages of an open machine.

3.1 The structure of the database

The tunnel characteristics (overburden, azimuth and diameter) are defined for each project, after that, the TBM performance database can be considered divided into two sections:

1) TBM specifications and TBM performance parameters.
2) Geological-geotechnical parameters of the rock mass.

Concerning the compilation of geological-geotechnical parameters, it is important to emphasise that they have been directly gathered from the tunnel projects or have been estimated using existing formulations commonly used in rock mechanics. The data quality varies from one project to another because the degree of detail is not the same and the information can have a quantitative and/or a qualitative nature.

3.1.1 TBM specifications and performance parameters

The information related to boring operations are included in the database, such as the bored length, the effective boring time and the total time needed to complete the excavation of a given tunnel section (Table 3.1).

Table 3.1 *Boring information, TBM specifications and TBM performance recorded for each section*

		Boring information	**TBM specifications**	**Recorded operational parameters**
Project name	*Section (first chainage-last chainage)*	Bored length [m] Total time [hr] Boring time [hr]	RPM_{max} [rev/min] Th_{max} [kN] Tq_{max} [kNm]	RPM [rev/min] Th [kN] Tq [kNm]

The TBM specifications are also reported in the database, as well as some TBM operational parameters such as the cutterhead rotation speed (RPM, rev/min), the thrust force (Th, kN) and the torque (Tq, kNm) recorded by the on-board acquisition system. These latter are called operational parameters, as they are varied by the TBM operator according to the encountered ground conditions.

Based on these data sets it has been possible to estimate other important parameters usually taken into account while describing TBM performance:

- the ratio between boring time and total shift time, i.e. the utilisation factor (U) of the TBM (Eq. 3.1, Table 3.2) which expresses the amount of time in which the machine has been effectively used for boring;
- the penetration rate (PR), in m/hr, defined as the distance excavated divided by the operating time during a continuous excavation phase (i.e. without considering time required for installing supports, TBM maintenance, etc.) (Eq. 3.2a and Eq. 3.2b, Table 3.2);
- the advance rate (AR), in m/hr, i.e. the TBM advance speed computed by considering the TBM delays due to rock supporting, maintenance, etc., calculated via the utilisation factor U and the penetration rate PR [34]. Commonly, the daily advance rate is reported (Eq. 3.3, Table 3.2);
- the field penetration index FPI [34, 39, 40] calculated as the ratio between average cutter load and attained penetration per revolution p, measured in mm/rev (Eq. 3.4, Table 3.2);
- the power consumed by machine (Eq. 3.5, Table 3.2);
- the net production rate (Eq. 3.6, Table 3.2);
- the specific energy needed to excavate a unit volume of rock (Eq. 3.7, Table 3.2).

Table 3.2 *TBM parameter equations in the TBM-performance database*

$U = \frac{Boring\ time\ [hr]}{Total\ time\ [hr]}$	(3.1)
$PR\ \left[\frac{m}{hr}\right] = \frac{Bored\ length\ [m]}{Boring\ time\ [hr]}$	(3.2a)
$p\ \left[\frac{mm}{rev}\right] = \frac{PR \times 1000}{RPM \times 60}$	(3.2b)
$Daily\ AR\ \left[\frac{m}{day}\right] = (PR \times U) \times 24$	(3.3)
$FPI\ \left[\frac{kN \cdot rev}{cutter \cdot mm}\right] = \frac{T_h}{N_c \times p}\ , N_c = number\ of\ cutters$	(3.4)
$Power\ [kW] = \frac{2\pi \times T_q \times RPM}{60}$	(3.5)
$Net\ Production\ Rate\ \left[\frac{m^3}{hr}\right] = \frac{RPM \times p \times 60 \times \pi \times \emptyset^2}{1000 \times 4}\ ,$ $\emptyset\ [m] = tunnel\ diameter$	(3.6)
$Specific\ Energy\ \left[\frac{MJ}{m^3}\right] = \frac{3.6 \times Power}{Net\ Production\ Rate}$	(3.7)

3.1.2 Rock mass parameters

For what concerns the rock mass characteristics, attention has been focused on the following parameters:

- Uniaxial Compressive strength, UCS [MPa]
- Brazilian Tensile Strength, BTS [MPa]
- Joint orientation
- Joint spacing
- Joint surface weathering conditions
- Volumetric Joints Count, J_v [joint/m^3]
- Rock Quality Designation, RQD
- Rock Mass Global Strength, UCS_{rm} [MPa]
- Measured water inflow
- Tangential stress, σ_ϑ [MPa]
- Geological Strength Index, GSI
- Q Index
- Rock Mass Rating, RMR

The intact rock properties for each lithotype, such as the Uniaxial Compressive Strength (UCS) and the Brazilian Tensile Strength (BTS), have been evaluated from laboratory tests on rock samples collected during excavation as well from detailed geological/geotechnical reports. The rock and joint conditions as well as the foliation attitude (e.g. orientation, spacing and surface conditions) have been gathered directly from face mappings, face/side-walls pictures and from tunnel profile surveys executed continuously during construction.

The volumetric joint count J_v, defined as the average number of joints in a cubic meter of rock, has been also calculated by considering the average spacing of the identified joint sets (Eq. 3.8, Table 3.3). Moreover, also based on J_v, an approximated Rock Quality Designation value RQD [79] has been computed (Eq. 3.9, Table 3.3).

Table 3.3 *Equations for rock mass parameters in the TBM-performance database*

$J_v\ [joints/m^3] = \sum\left(\frac{1}{s_i}\right) + \left[\frac{N_{r(5)}}{5}\right]$ $s_i\ [m] = average\ spacing\ of\ the\ i^{th}\ joint\ set$ $N_{r(5)} = number\ of\ random\ joints\ along\ a\ 5\ m\ scan\ line$	(3.8)
$RQD\ [\%] = 100e^{-0.1\lambda}(1+0.1\lambda)$ $\lambda = total\ joint\ frequency$	(3.9)
$UCS_{rm}\ [MPa] = UCSs^a$	(3.10a)
$s\ [-] = \frac{\exp(GSI-100)}{9-3D}$ with $D = 0\ (TBM\ tunnelling)$	(3.10b)
$a\ [-] = \frac{1}{2}+\frac{1}{6}\left(e^{-GSI/15}-e^{-20/3}\right)$	(3.10c)
$UCS_H\ [MPa] = tUCS$	(3.11a)
$t\ [-] = \left(\frac{GSI}{100}\right)^{0.55}$	(3.11b)

The recorded information about water inflows, as described in the geological reports of the projects or from the tunnel surveys, is also included in the database.

In addition, the Rock Mass Rating (RMR), the Q index and the Geological Strength Index (GSI) are computed (or evaluated) on the basis of all above collected data.

The rock mass uniaxial compressive strength (UCS_{rm}) [80] is computed with Eq. 3.10a. The uniaxial compressive strength of intact rock (UCS) is reduced by the Hoek and Brown parameters, s and a, which depend on the GSI index (Eq. 3.10b and Eq. 3.10c). The "switch" value GSI = 25 in the calculation of the coefficients s and a (see Chapter 2, paragraph 2.3, Table 2.7), has been eliminated for Equations 3.10b and 3.10c that give smooth continuous transitions for the entire range of GSI values [81]. The factor D depends on the degree of disturbance due to blast damage and stress relaxation: it varies from 0, for undisturbed in situ rock masses, to 1, for very disturbed rock masses. In the present case, a value of D = 0 has been considered (TBM excavation).

Finally, in order to better take into account the altered characteristics of the cataclastic and weak rocks, the strength parameters calculated as proposed by Habimana [8, 76] have been included in the database. As reported in Chapter 2 (see paragraph 2.3, Table 2.8), the uniaxial compressive strength of the weathered material will be reduced according to t (Eq. 3.11a), where t is a coefficient depending on GSI that varies between 1, in case of intact rock, and 0, in case of a soil-like material (Eq. 3.11b).

3.2 Data quality

According to the different nature of the sources, a preliminary analysis has been done to show the distribution of compiled data as well as their quality.

In Tables 3.4 and 3.5 the sources of TBM performance and geological-geotechnical characteristics are reported.

As it can be clearly seen in Table 3.4, the information about the performance of the TBM has been the hardest one to obtain. The calculation of the basic machine parameters has been generally performed on the basis of the measures carried out by the TBM on-board acquisition system during construction. However, it is important to underline that it was not possible to gain complete TBM data for all tunnel projects (up to 69% of "no data" for one of the selected parameters). As a matter of fact, the contractors are often unable to share easily the data from the site especially when on-going claims have not been resolved yet.

Although important TBM performance parameters (such as the penetration per revolution, thrust force and torque) have not been collected for all projects, the daily advance rate (meters per day) of each section has been included in the database. This has been possible thanks to the available time-way diagrams and to the information found in literature about the delays and the stoppages of the machine in difficult ground conditions for each specific case studied.

Despite the difficulties in gaining complete TBM data, enough information related to the boring process and to the TBM advancement has been collected.

Regarding the characteristics of the rock mass, several and more detailed sources have been considered:

- Geological-geotechnical reports;
- Laboratory tests;
- Stereographic projections;
- Tunnel profile surveys;
- Face and side-wall pictures;
- Literature.

It is important to underline that geological-geotechnical descriptions of tunnel projects are easier to obtain than TBM performance data. As a matter of fact, they belong to the tunnel owner and not to the contractor and, also for this reason, they are more simply achievable.

Table 3.4 The different sources of TBM performance

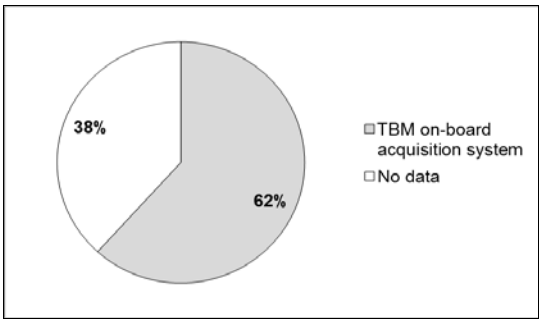
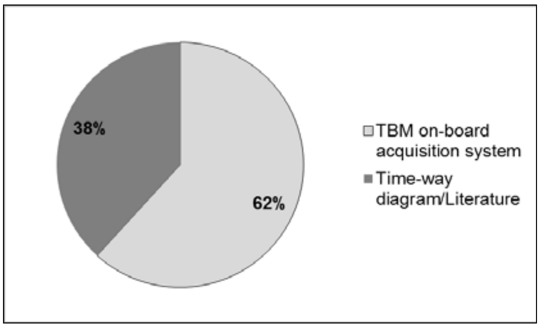
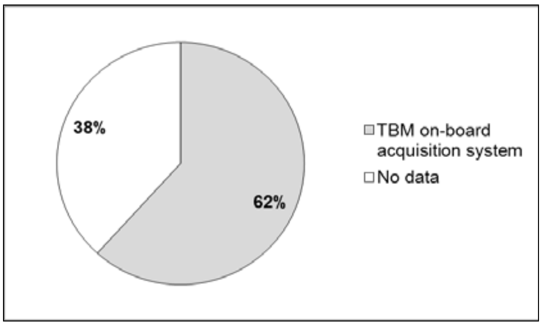
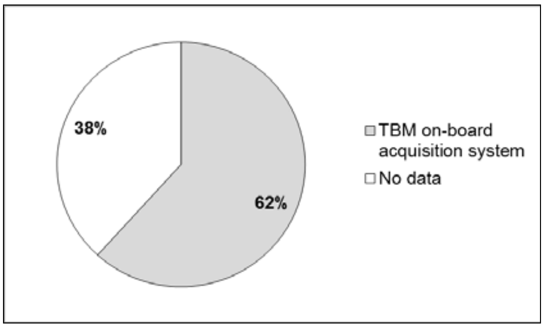
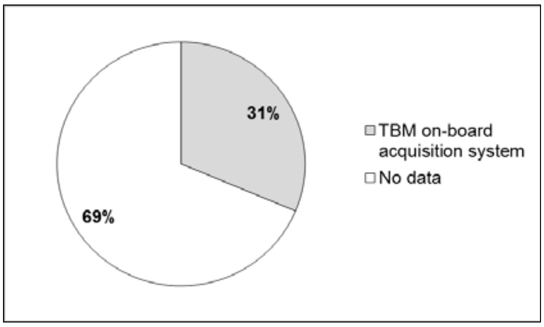
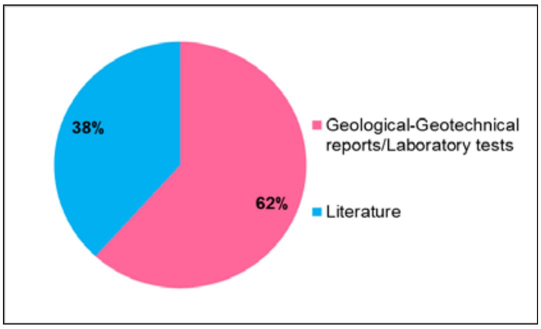
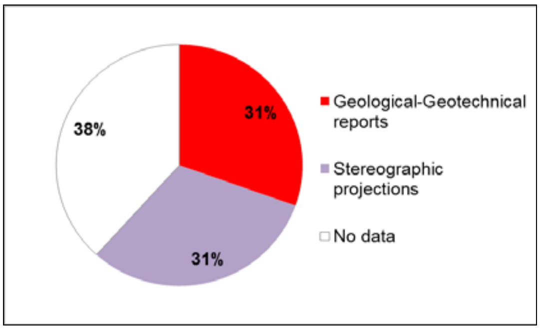
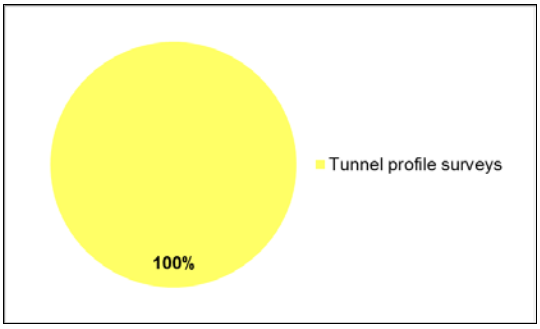
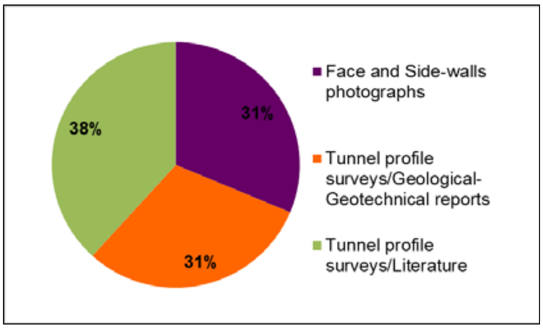
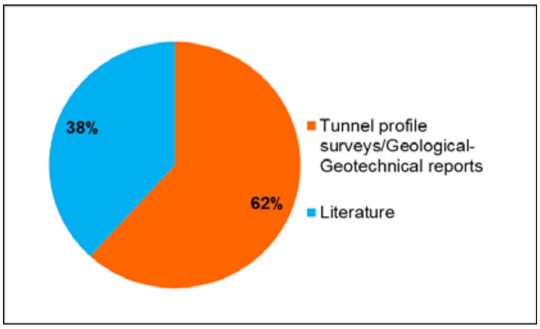
	Penetration Rate
	Advance Rate (delays)
	RPM
	Thrust force
	Torque

Table 3.5 The different sources of geological-geotechnical parameters

	Rock strength
	Joint orientation
	Joint spacing
	Joint surface/Rock weathering conditions
	Water inflows

Except for the joint orientation (38% of data were not available), all parameters have been collected for each tunnel projects, but with a variable detail and reliability. If the attention is focused on the “rock strength”, for instance, it is evident that the literature has played an important role. This is because laboratory test results have not been always available and in extremely challenging environments, as fault zones and highly fractured rocks, the tests were not even possible. In such situations, based on the knowledge of the lithotype of the host rock, the ranges of strength reported in literature have represented the reference point.

3.3 Selection criteria for data collection

A specific criterion has been adopted in order to select the tunnel sections to be included in the TBM-performance database.

Attention has been focused only on stretches identified as difficult/very difficult ground conditions, such as highly fractured or faulted rocks, over a length of 8-10 m at least.

In order to select tunnel sections in these challenging environments, RMR values lower than 40 have been considered. As a matter of fact, according to the classification provided by Bieniawski [82], if RMR is lower than 40, the rock mass quality varies from poor (IV class) to very poor (V class). Despite not all the geological-geotechnical parameters were available with the same degree of detail, the RMR value has been computed on the basis of specific considerations concerning:

- water inflow;
- fracturing degree of rock mass and joint spacing;
- weathering conditions of both the matrix and joint surfaces;
- support profile;

The selected zones are generally also characterised by large/very large water inflows (> 60 l/min) that often contribute to lower values of RMR.

For what concerns the joint spacing (s), five classes have been considered (Table 3.6) and only the sections belonging to classes C, D and E have been included in the database. When only a qualitative description of the rock mass conditions was available (i.e. from “slightly fractured” to “disintegrated” in Table 3.6) sections characterised by “Highly Fractured” (corresponding to classes C and D) and “Disintegrated” (corresponding to class E) rocks have been included in database.

***Table 3.6** Quantitative and qualitative description of fracturing degree*

Fracturing degree class ***(according to the quantitative description)***	**Quantitative description** ***Joint spacing, s [m]***	**Qualitative description**
A	s > 5	Slightly fractured
B	1 < s < 5	Fractured
C	0.1 < s < 1	Highly fractured
D	s ≤ 0.1	
E	s ≤ 0.01	Disintegrated

Concerning the weathering degree of the rock and joint surfaces, to analyse and interpret the collected data was not an easy task. As a matter of fact, the characteristics of the joints (roughness, opening, alteration/filling) are not always clear or well-defined, especially considering extremely fractured rock masses where it is no more possible to identify the discontinuities. In those cases the rock conditions become the most relevant information. The alteration and the weakness of the material can be the result of deterioration due to faulting and folding processes. According to these considerations, in Table 3.7 the quality of the rock mass is described through different characteristics:

- the behaviour of the rock;
- the rock weathering;
- the rock strength.

Based on the nature and on the detail degree of the available data, one of the three criteria has been selected in order to associate the class to the rock.

Table 3.7 *State and behaviour of the rock*

Class	Rock behaviour	Rock weathering	Rock strength (UCS, MPa)
1	Good	Unweathered	Very strong (> 100)
2	Fair	Slightly weathered	Strong (60-100)
3	Mediocre	Weathered	Medium strong (40-60)
4	Very poor	Highly weathered	Weak (< 40)

In general, very fractured and disintegrated rocks have been included in the database, therefore, the volumetric joint count (J_v) is always rather high. However, there are sections with lower values of J_v and this is due to the fact that other factors (such as the weathering degree or water inflows) have played a more important role in the choice of the sections. For instance, rock masses characterised by an important alteration of the rock and/or of the joint surfaces are not necessarily described by high J_v.

Finally, also the available information about the (temporary) supports has been taken into account as an additional description of the rock mass conditions and of the problematic zones encountered during construction. Only sections with heavy/very heavy rock supporting measures have been considered.

3.4 Range of variation of significant parameters

Among the compiled data significant parameters concerning TBM specifications (p, daily AR) and geological-geotechnical characteristics (J_v, GSI and UCS_H) have been selected in order to study their range of variation in the database.

3.4.1 TBM performance parameters

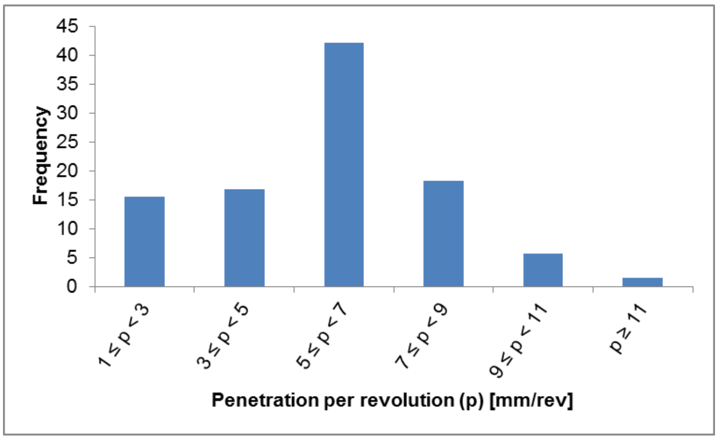
The choice of the TBM parameters based on the fact that the penetration per revolution (p) and the daily advance rate (daily AR) allow evaluating the performance of the machine with or without possible delays due to installing supports, TBM maintenance etc.

The penetration per revolution (p) is characterised by a range of variation comprised between 1 mm/rev to more than 11 mm/rev, with a mean of 5.76 mm/rev and a mode equal to 5.98 mm/rev (Figure 3.1). These rather high recorded values are due to the significant rock mass degree of fracturing that improves TBM performance in terms of penetration. This is because the crushing and chipping processes are favoured by the discontinuities which enhance fracture propagation and interaction beneath the disc cutters.

It is important to underline that the higher values of p (> 9 mm/rev), recorded during construction, can be assumed negligible for the purposes of these analyses.

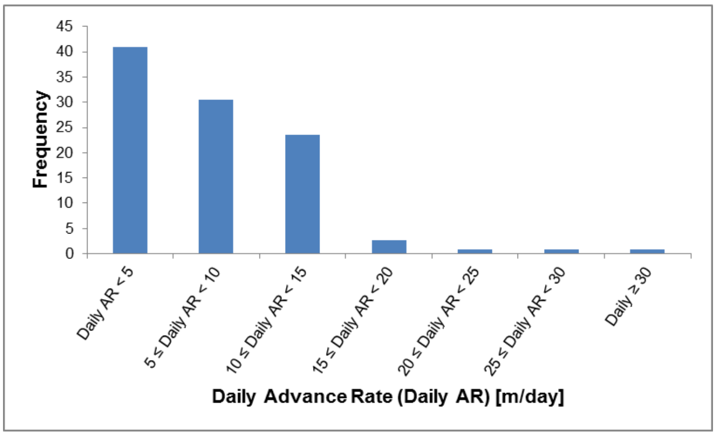
The potential delays occurring during excavation have repercussions on the daily advance rate of the machine and are accounted via the utilisation factor U (Eq. 3.3, Table 3.2). As already mentioned, it includes downtimes due to increasing rock support requirements, additional drainage systems, gripper bearing failure etc.

As a matter of fact, the obtained daily advance rates show an important lowering compared to the penetration rates (Figure 3.2). The range of variation includes values lower than 5 m/day (down to performance lower than 1 m/day) as far as values greater than 30 m/day, with a mean of 7.31 m/day and a mode in the range 5-15 m/day. More than 90% of the tunnel sections show a daily advance rate lower than 15 m/day and about half of them do not reach 5 m/day. The higher values (especially greater than 20 m/day) correspond to percentages so low to be inconsiderable. The large data scatter, expressed by a high coefficient of variation (C.V. = 0.76), underlines the high uncertainty related to the estimation of the TBM utilisation factor which involves both geological/geotechnical and construction-related parameters. Despite machine delays mainly depend on the TBM-rock mass interaction, they can indeed be affected also by unpredictable events (often due to the human factor) which is almost impossible to quantify.



Mean p [mm/rev]	5.76
Standard deviation [mm/rev]	2.16
C.V.	0.37

Fig. 3.1 *Histogram plot for the penetration per revolution (p) of the TBM with indication of mean, standard deviation and coefficient of variation (C.V. = standard deviation/mean).*



Mean Tot AR [m/day]	7.31
Standard deviation [m/day]	5.56
C.V.	0.76

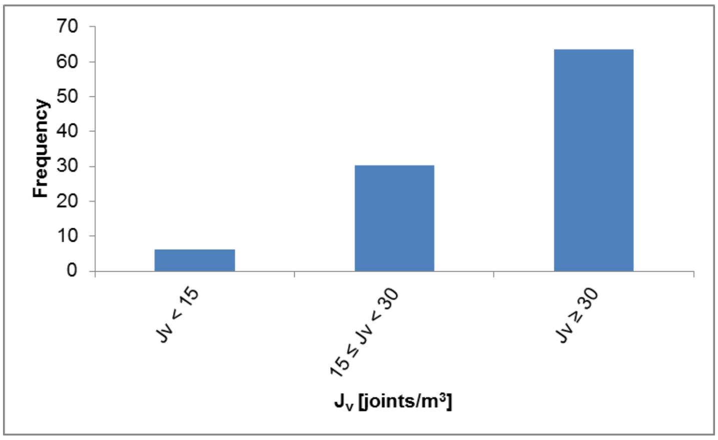
Fig. 3.2 *Histogram plot for the daily Advance Rate (AR) of the TBM with indication of mean, standard deviation and coefficient of variation (C.V. = standard deviation/mean).*

3.4.2 Rock mass parameters

For what concerns the geological-geotechnical characteristics the degree of fracturing (expressed by J_v) has been considered as one of the most representative parameter, as well the weathering conditions of the rock matrix and of the joint surfaces (considered in the definition of GSI and UCS_H).

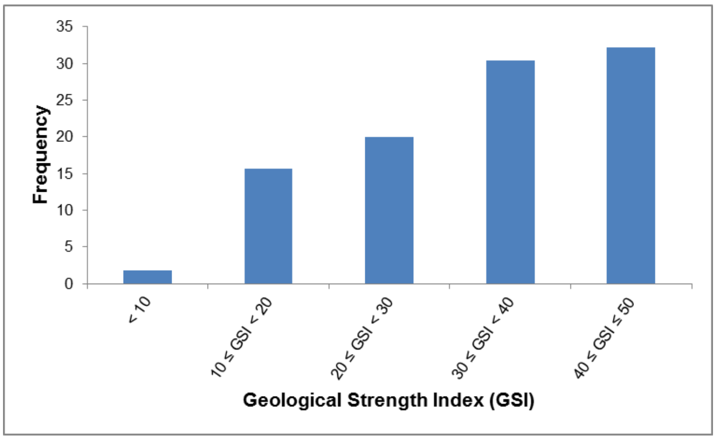
The range of J_v is comprised between values lower than 15 joints/m^3 and values greater than 30 joints/m^3 (Figure 3.3). However, the mode (J_v > 30 joints/m^3) and the mean (about 28 joints/m^3) underline the generally high (and extremely high) fracturing degree which characterises the rock masses of the sections included in the database. The highly jointed structure contributes, together with the conditions of the joint surfaces, to rather low values of GSI (Figure 3.4).

Concerning the Geological Strength Index the range varies between 5 and 50 in the database (Figure 3.4), with a mode value in the range 30-40 and a mean value of about 30. According to the modified version of GSI [73, 74], values lower than 40 describe strongly disturbed and folded rock masses, characterised by important degree of tectonisation and faulting as far as disintegrated material.



Mean J_v [joints/m^3]	27.77
Standard deviation [joints/m^3]	9.33
C.V.	0.34

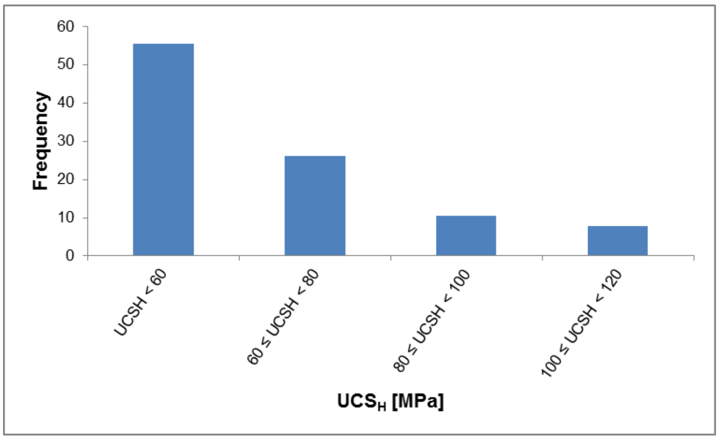
Fig. 3.3 *Histogram plot for the volumetric joint count (J_v) with indication of mean, standard deviation and coefficient of variation (C.V. = standard deviation/mean).*



Mean GSI	29.00
Standard deviation	11.24
C.V.	0.39

Fig. 3.4 *Histogram plot for the Geological Strength Index (GSI) with indication of mean, standard deviation and coefficient of variation (C.V. = standard deviation/mean).*

The rock strength estimated according to Habimana [8, 76] covers a range between 10 MPa and 120 MPa (Figure 3.5), with a mean and a mode value of 58.2 MPa and 36.7 MPa respectively. More than 50% of tunnel sections are characterised by rock with a reduced strength lower than 60 MPa (33% with UCS_H lower than 40 MPa and about 22% with UCS_H between 40 MPa and 60 MPa). These values correspond to weak and medium strong rocks (see Table 3.7) while about 36% and only 8% refer to rocks defined respectively as strong and very strong. Several rock types included in the database (schist, limestone, gneiss, greenstone, granite etc.) justify the large range of UCS_H that is affected by different values of uniaxial compressive strength characterising the different (intact) rocks. The strength of the altered rocks appears to be extremely reduced because of the high fracturing and/or weathering degrees of the material expressed by low values of GSI.



Mean UCS_H [MPa]	58.25
Standard deviation [MPa]	23.29
C.V.	0.40

Fig. 3.5 *Histogram plot for the Uniaxial Compressive Strength of the rock (UCS_H) calculated by Habimana [8, 76] with indication of mean, standard deviation and coefficient of variation (C.V. = standard deviation/mean).*

4 Preliminary data analyses

Based on the selection criteria reported in Chapter 3, more than 100 tunnel sections characterised by faulted and fractured rocks have been included in the TBM-performance database. The parameters commonly adopted in the existing TBM-performance prediction models have been considered in order to investigate the correlations among the rock mass characteristics and TBM advancement and rate of penetration. In order to proceed with preliminary analyses on the database, the uniaxial compressive strength (UCS) and the volumetric joint count (J_v) have been selected as representative parameters for the material strength and for the fracturing degree of the rock according to what reported in Chapter 1 (paragraph 1.5).

4.1 Rock strength: correlations between UCS (and UCS_H) and TBM performance parameters

Based on the literature information, the strength values (Figure 4.1) reported in the database indicate strong and very strong rocks (UCS = 60-180 MPa). These values do not represent the weak material that should characterise the rock mass of tunnel sections under consideration for the analyses. For this reason the already introduced uniaxial compressive strength reduced by Habimana [8, 76], which takes into account the weathering degree of rock mass, has been selected for evaluating a possible correlation between rock properties and TBM performance. As a matter of fact, it should be reminded that UCS_H depends on both rock and joint surfaces alteration as expressed by the Geological Strength Index (GSI).

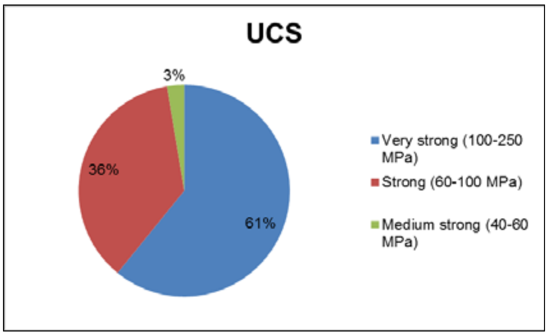


Fig. 4.1 *Classes of strength (Uniaxial Compressive Strength, UCS) of the rocks included in the database.*

Despite a significant lowering of strength values (that become more representative of the real conditions, see Figure 4.2), a large data scattering remains and the correlation with both p and PR is almost meaningless (Figures 4.3 and 4.4).

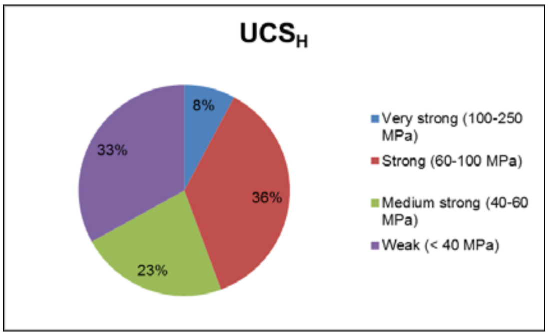


Fig. 4.2 *Classes of reduced strength (Uniaxial Compressive Strength, UCS_H, for Habimana, [8, 76]) of the rocks included in the database. The same results have been already shown in Figure 3.5 (see Chapter 3), where different intervals of UCS_H have been considered.*

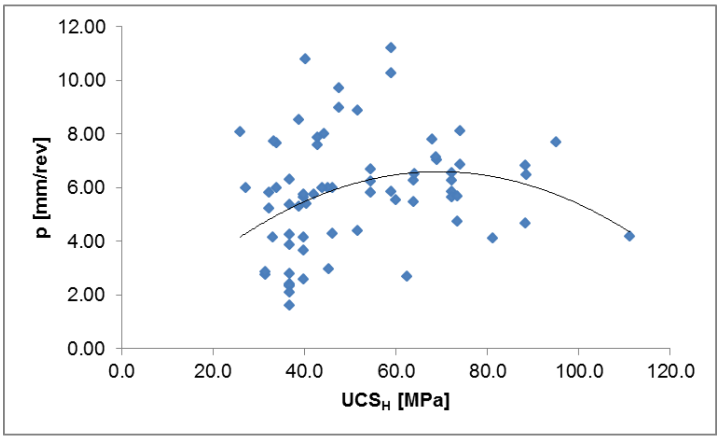


Fig. 4.3 *Correlation between penetration per revolution (p) and the Uniaxial Compressive Strength reduced by Habimana [8, 76] (UCS_H).*

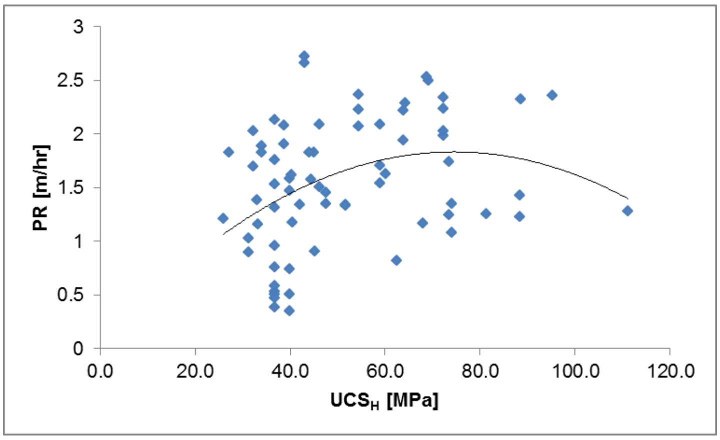


Fig. 4.4 *Correlation between Penetration Rate (PR) and the Uniaxial Compressive Strength reduced by Habimana [8, 76] (UCS_H).*

4.2 Fracturing degree: correlations between J_v and TBM performance parameters

Regarding the fracturing degree of the rock mass, the first analysis investigated the relationship between the penetration per revolution (p) and J_v. Figure 4.5 shows how an increasing degree of fracturing improves TBM performance in terms of penetration. In this case the best fitting of the data is provided by a polynomial curve. Although the accuracy of the relationship is not very high due to the large scattering of data, a general increase of p for increasing J_v may be observed. This can be explained by the fact that the discontinuities are planes of weakness in the rock mass, and they assist the crushing and chipping of the rock during excavation, beneath and adjacent to the cutters [83, 84]. For this reason, a high number of joint sets, generally does not represent a hindrance to the boring process, but, on the other hand, it can also generate other difficulties mainly linked with face/wall stability problems and increasing TBM downtimes. However, it should be noticed that once a certain threshold of rock mass fracturing is exceeded, its effect on the TBM advance starts to decrease. In Figure 4.5 this behaviour is represented by the downward concaveness of the fitting curve.

In Figure 4.6 the correlation between PR and J_v is reported. Compared to p, PR undergoes a more evident decrease for values greater than 30 joints/m^3. As a matter of fact, while p refers to the penetration for one cutterhead revolution, PR is affected by the rotational speed (RPM) which in its turn undergoes a reduction for significant rock mass fracturing degrees. This reduction is probably due to the fact that, in highly fractured rocks, other operational issues, such as the efficiency of the muck removal system to evacuate the excavated material from the face, may affect the applicable RPM and, therefore, achievable rate of penetration [85].

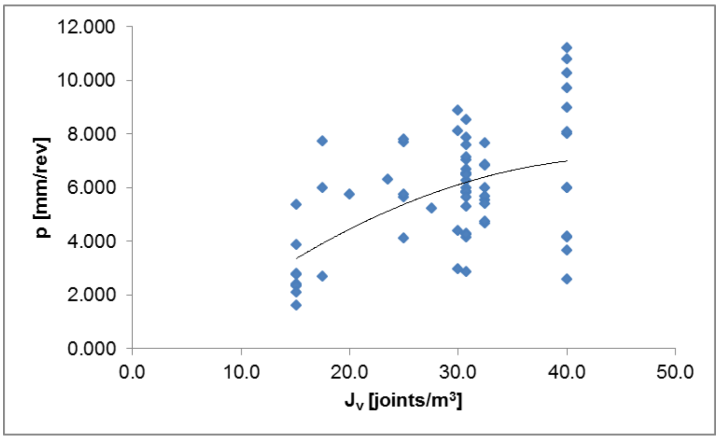


Fig. 4.5 *Correlation between penetration per revolution (p) and Volumetric Joint Count (J_v).*

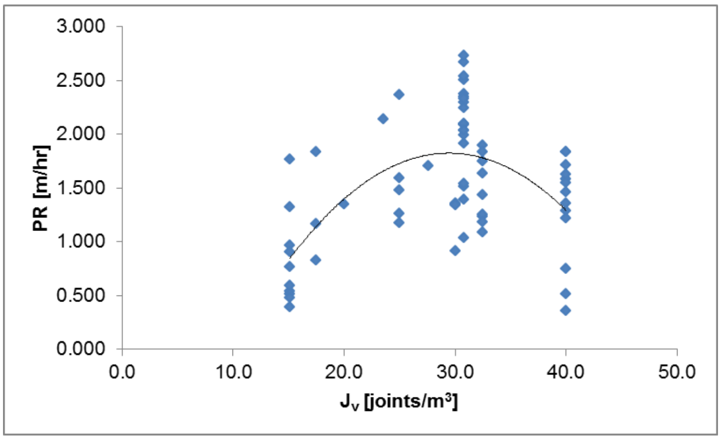


Fig. 4.6 *Correlation between Penetration Rate (PR) and Volumetric Joint Count (J_v).*

In the preliminary analysis of the collected data, in despite of good penetration rates (i.e. PR between 0.38 and 2.73 m/hr), the TBM utilisation factor (U) was frequently very low, reaching extreme values of only 3%. The obtained advance rates (based on Eq. 3.3), including downtimes for TBM maintenance, machine breakdown and tunnel failures, could be consequently extremely low (i.e. about 1.0 m/day). The computation of the potential TBM delays and the prediction of the utilisation factor of the machine and the advance rate can be considered within the most important issues when back-analysing the performance of TBMs in difficult ground conditions.

4.3 Existing rock mass classification systems: correlations between RMR and TBM performance parameters

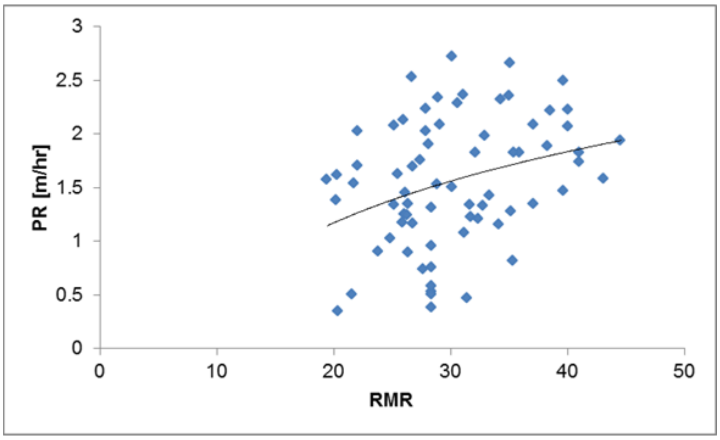
Analyses have been performed for investigating existing relationships between RMR and both PR and AR (Figures 4.7 and 4.8).

The Rock Mass Rating, providing a general geomechanical description of the rock mass, has been selected as reference index because of the difficulties in finding satisfying relationships among TBM performance and individual geological-geotechnical parameters.

Together with the raw information (Figures 4.7a and 4.8a), data have been grouped in 5 RMR classes and plotted as bar charts: the central point of the bars represents the average value of PR (Figure 4.7b) and total AR (Figure 4.8b) for each class, while the bar length indicates the standard deviation.

As it can be observed in Figures 4.7 and 4.8, respectively for PR and AR, although a quite large data scattering exists, a certain trend can be identified: if RMR increases both average PR and total AR also increase.

(a)



(b)

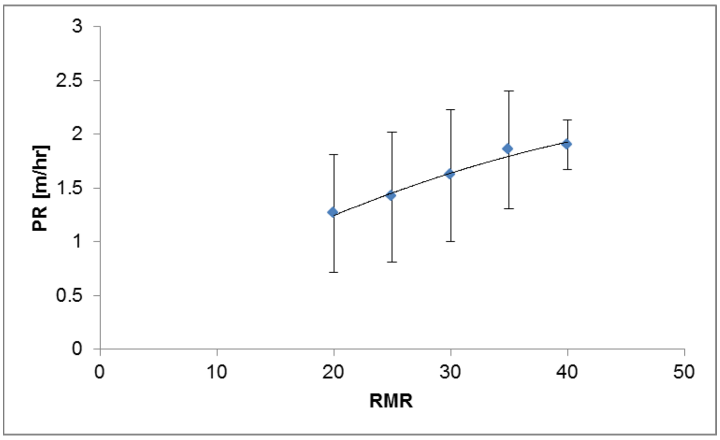
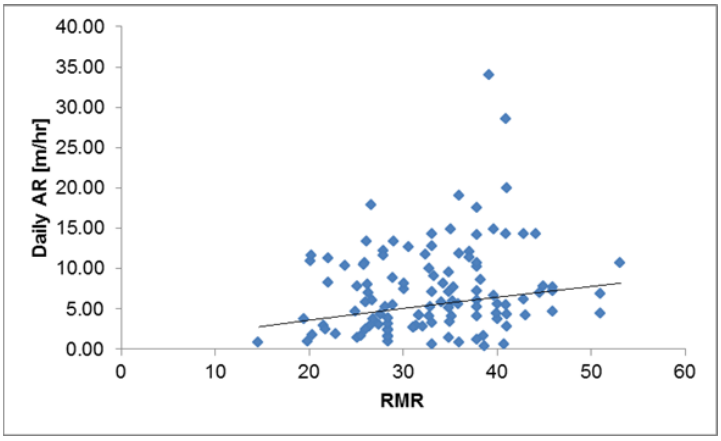


Fig. 4.7 *a) Correlation between Penetration Rate (PR) and Rock Mass Rating (RMR) (raw data); b) correlation between average PR and average RMR in each class.*

The preliminary results seem to confirm the trend reported in Sapigni et al. [35]. Since only sections characterised by RMR lower than 40 have been included in the database, only the increasing of PR with increasing of rock mass quality could be investigated (which corresponds to the left part of the bell shape curve represented in Figure 1.11). The scatter around the mean value is rather high and it is probably related to the difficulty in maintaining a constant thrust during excavation [35].

The correlation between total AR and RMR follows the same trend. The reason of that is twofold: in one hand, the PR increases for increasing RMR and, at the same time, also an increase of the TBM utilisation factor (U) can be assumed due to better conditions for excavation. In an attempt to find a possible relationship between U and RMR, a high data scatter has been observed [86]. This is mainly due to the fact that the Rock Mass Rating depends on several factors (fracturing degree, water inflow, rock strength, joint weathering conditions) and each of them might influence the utilisation factor with a different weight that should be considered according to the particular encountered conditions. In addition, it is necessary to remind that the conventional RMR system has been developed to provide support guidelines for underground construction excavated by the Drill-and-Blast method. This means that it does not take into account rock-machine interaction parameters, and therefore, it could be inadequate for TBM performance prediction.

(a)



(b)

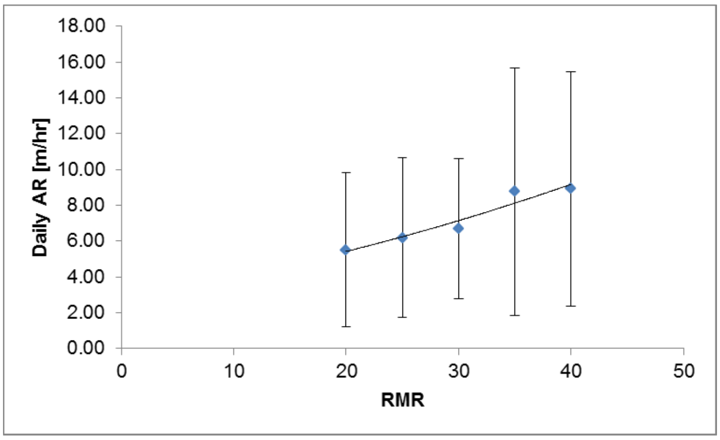


Fig. 4.8 *a) Correlation between Advance Rate (AR) and Rock Mass Rating (RMR) (raw data); b) correlation between average AR and average RMR in each class.*

4.4 Final remarks

The preliminary results reported in this section highlight the great difficulty in characterising the degraded and altered rock mass because of the extreme complexity in identifying and quantifying the relevant countless factors.

If the attention is focused on the potential relationships between the strength of rock (UCS) and TBM performance, the probability of finding a correlation is rather low. As reported before, a fault zone can be described as a mixture of soil and rock components, therefore the properties of the intact rock may lose importance in the characterisation of the material. Other factors, such as fracturing degree and water inflow, might become more relevant in order to study possible relationships with the TBM advancement. It is also necessary to underline that the values of UCS included in the database are partly collected from the literature, therefore, their reliability is limited.

In order to take into account the alteration undergone by the rock material, the reduced strength UCS_H has been thus considered, but the attempt to find possible correlations with the TBM parameters did not provide satisfying results.

The introduction of a “more global” characterisation of fault and highly fractured zones may be necessary in order to try to correlate TBM performance parameters with geomechanical behaviour of the rock mass. With this purpose, referring to the existing rock mass classification systems seemed to be a valid starting point. The analyses performed considering the Rock Mass Rating (RMR), showed agreement with previous works reported in literature. However, the RMR system proved to be still not appropriate to characterise difficult ground conditions with the aim to predict the TBM performance.

Furthermore, it is important to point out that some basic factors which are basic for RMR definition, (e.g. the rock strength) are extremely significant for predicting performances in hard rock tunnel boring, but they may play a very marginal role in the definition/estimation of the machine performance in highly fractured and faulted zones.

As a consequence, it is important to better characterise the fault zones, trying to classify them on the basis of several significant geological/geotechnical parameters which can be directly connected with TBM performances and potential construction delays.

5 Fault zone classification and correlations with TBM performance

Because of very complex structure and nature of the fault zones and their difficult geomechanical characterisation, it seems necessary to develop a specific classification for these particular environments.

In this framework, the term “fault zone” basically refers to the difficult ground conditions encountered in TBM tunnelling, characterised by extremely jointed rock masses or faulted/sheared rocks.

The most common and approved terminology for the fault rocks has been adopted to describe each group of the proposed classification. In order to identify the “fault zone” classes the following aspects have been firstly taking into consideration:

- The fracturing degree of the rock mass;
- The weathering degree of the rock mass;
- The possible influence of the water;
- The effect of the depth (in-situ stress).

5.1 Significant geological/geotechnical parameters in the classification

The fracturing degree of the rock mass and the weathering degree of the rock and of the joint surfaces represent basic parameters for defining the “fault zone” classes. While potential water inflows and depth of faulted/fractured zones can be considered only as additional factors which contribute to the reduction of TBM performance independently of the class. The following sections describe in more detail the influence of all these aspects.

5.1.1 The fracturing degree

The blockiness of the rock mass could represent a valid base of departure to identify the different classes and the volumetric joint count (J_v) has been selected as the most appropriate parameter to represent the degree of fracturing. J_v is generally given as a range and it is an average measurement of the number of joints that intersect a volume of the rock mass. The ISRM [87] proposed a description of J_v in terms of:

- very large blocks: J_v < 1 joints/m^3;
- large blocks: 1 < J_v < 3 joints/m^3;
- medium-sized blocks: 3 < J_v < 10 joints/m^3;
- small blocks: 10 < J_v < 30 joints/m^3;
- very small blocks: J_v > 30 joints/m^3.

In the framework of this study, the proposed fault zone classification only blocks with small to very small size have been considered, adopting J_v equal to 10 joints/m^3 as lower limit, below which the rock mass cannot be described anymore as highly fractured or faulted. Regarding the upper limit, the exact definition of the threshold beyond which the rock is completely crushed is not immediate. For this reason a reference value of 35 joints/m^3 has been considered as the highest volumetric joint count for a rock mass where individual joint sets can be still identified.

5.1.2 The weathering degree

Another important factor involved in the definition of the “fault” classes is the weathering degree of the rock and of the discontinuity surfaces. In order to take into account this aspect, the existing classification systems for weak and weathered rocks have been considered to describe the different grades of alteration suffered by the rock mass and their effects on some engineering properties.

In order to have a qualitative characterisation of the rock mass included in the classification of highly fractured and faulted zones, it was been decided to refer to the Geological Strength Index (GSI).

The GSI is a system of rock mass-characterisation [88, 89] used to determine the rock mass strength and the rock mass deformation modulus. This system provides an estimation of the rock mass properties based on qualitative descriptions of the blockiness of the mass and of the conditions of the joint surfaces. It has been modified over the years [72-74] in order to be adopted in different geological situations, such as sheared rock masses characterised by a very poor quality and heterogeneous and tectonically disturbed lithological formations.

The general chart for estimation of the Geological Strength Index (GSI) for jointed rock masses is presented in Figure 5.1.

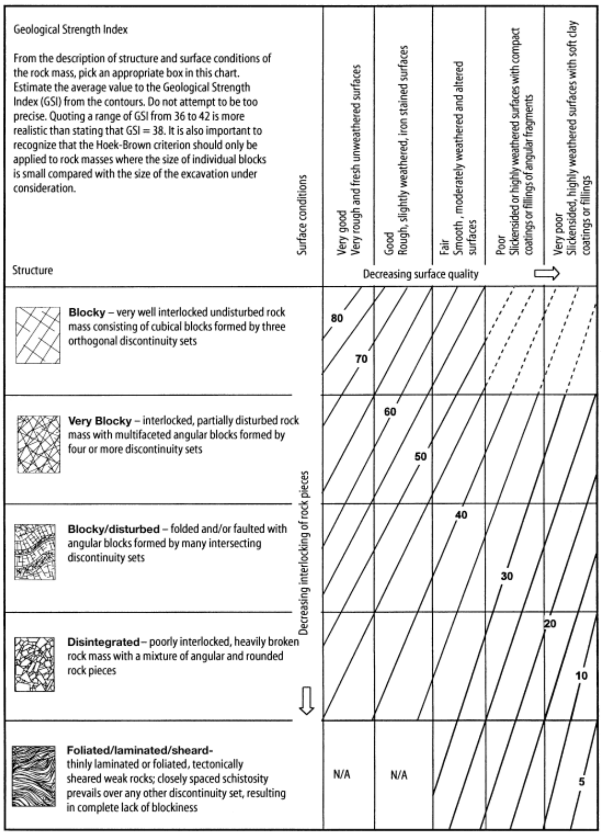


__Fig. 5.1__ The Geological Strength Index for jointed rock masses [72].

The “foliated/laminated/sheared” class shown in the bottom row of the chart, consists of rock masses of non-blocky structure with a low strength and a high deformability, where a well-defined persistent and closely spaced lamination (or foliation) is dominant with respect to any other discontinuity set and shows slickensided surfaces characterised by an important weathering degree. Unlike the disintegrated category, where the rock-to-rock contacts govern strength and deformability, the mechanism of deformation in foliated/laminated/sheared class is controlled by the displacements along the foliation planes and by the shear strength of the fines along the surfaces [72].

The description of foliated/sheared category in the GSI system is very close to the structural and geological characteristics taken into account to define the classes of the highly fractured and faulted rocks classification developed for this work. In particular, the pre-sheared nature of the rock mass is a condition easily encountered in fault zones.

Based on these considerations, extremely low values of GSI (down to 5) have been considered in the characterisation of the identified classes.

In order to take into account the complex nature of a fault zone, often constituted of a combination of material with different characteristics of strength and deformability (rock and soil-like materials), the more recent Geological Strength Index, developed for heterogeneous rock masses [73, 74], is particularly interesting because it has been introduced to describe rock masses, such as flysch, composed of frequently tectonically disturbed alternations of strong and weak rocks [90]. This new version of the GSI (Figure 5.2) refers to the composition and structure of the mass, in terms of tectonic disturbance and alternation of the weaker layers, rather than to the blockiness and interlocking of rock pieces, while the surface conditions of the discontinuities predominantly refer to the bedding planes.

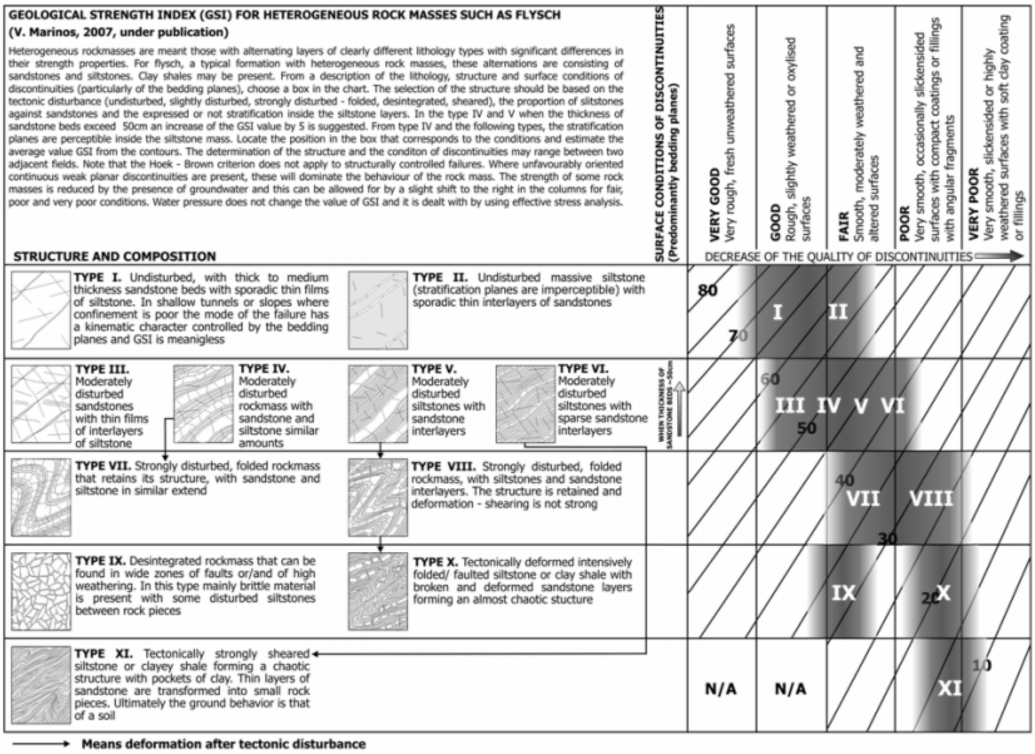


Fig. 5.2 *The Geological Strength Index for heterogeneous rock masses such as flysch [73].*

On the basis of this new chart for the GSI a tool for a more detailed description of the weak and weathered rock masses, where the tectonic disturbance plays such an important role, has been provided.

The classification of highly fractured and faulted zones has made reference to this system in order to describe the classes, especially about the altered mass structure due to faulting and folding activities.

5.1.3 The water influence (rock mass permeability)

The rock permeability can be influenced not only by the primary porosity (the original percentage of void space within the rock) but also by the fracturing degree. As a matter of fact, a highly fractured rock mass could be more prone to important water inflows than a very porous soil that is characterised by a lower permeability.

The behaviour and the response of the rock mass to external actions are highly influenced by the potential water inflows. The possible amount of water may vary from low to extremely high in all the identified classes and it is linked to the permeability and porosity which are characteristic parameters of the rock mass. Thus a fault zone may behave as a conduct or as a barrier and in some cases as a combined system depending on the fracture network and on the amount of generally less permeable fine-grained matrix [52]. The weathering degree of the rock (and of the joint surfaces) could be a reference parameter in order to describe the potential fluid flow in a fault zone; the products of rock alteration (generally fine-grained material) could indeed significantly reduce the permeability.

The rheology of the host rock can influence the hydrological properties of a fault zone. As already reported in Chapter 2, the brittle deformation of competent and incompetent rocks leads to different permeability characteristics of the crushed material. According to the results of Sausgruber and Brandner [60], brittlely deformed incompetent host rocks create zones with soil-like features, thus characterised by a low permeability. On the contrary, brittlely deformed competent host rocks create nearly non-cohesion zones which are extremely permeable and characterised by a high potential of water inflow.

5.1.4 The depth influence (in-situ stress)

In normal conditions the depth (by influencing the in-situ stress) plays an important role in the definition of potential risks and stability issues in tunnelling. The uncertainties increase with depth, greater efforts and costs are therefore necessary for further site investigations in order to foresee the hydrological and geotechnical conditions. Problematic events are indeed more frequent in weakness zones such as heavily tectonised and fractured rocks characterised by poor mechanical and anisotropic conditions. In Figure 5.3 the rock failure mode matrix [91] shows the combined effect of in-situ stress and rock mass conditions on the behaviour of the rock mass and on failure mode near underground excavations. The weak and highly fractured rocks are highlighted in grey (RMR < 50).

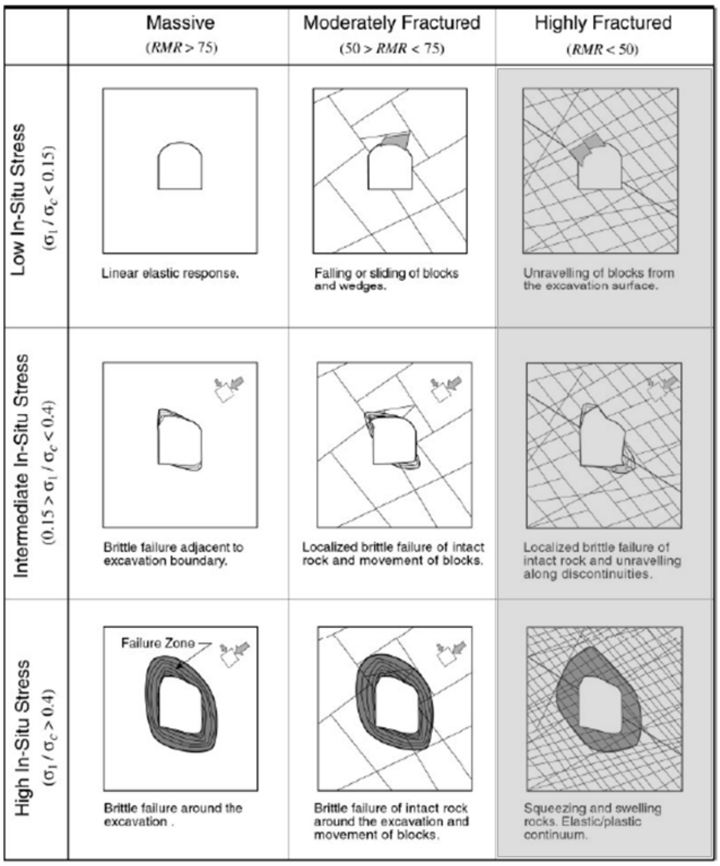


Fig. 5.3 *Rock failure mode matrix [91], where σ_c is the unconfined compressive strength of intact rock and σ_1 is the maximum far-field stress.*

In rocks of poor quality the deeper the tunnel, the higher is the squeezing potential. Moreover, also the probability of problems related to groundwater (huge water inflows and/or significant water pressure) shows a significant increase for deep tunnels.

5.2 Description of "fault zone" classes

As already mentioned in the previous paragraph, the fracturing degree and the weathering degree of the rock mass have represented the main factors in the characterisation of the "fault zone" classes. On the contrary, the potential water inflows and the influence of tunnel depth represent only complementary parameters since they can affect tunnelling operations (thus the TBM performance) also outside from a fault zone.

The classes have been identified mainly based on a detailed literature review of existing classification systems developed for weak rocks (see also Chapter 2).

In this framework, the goal is to propose a classification that allows estimating the potential correlations existing between performance of the TBM and the geomechanical behaviour of each identified "fault zone" class.

5.2.1 I class: highly fractured rock masses

The first class consists of rock masses characterised by a high fracturing degree and by no visible sign of weathering of the rock material. The original fresh rock and the discontinuity surfaces could be affected by a slightly discolouration, but the geomechanical properties of the intact rock are not subject to any alteration.

The "mechanical weathering" (fracturing and potential opening of the joints) is therefore the only involved process, while the "chemical weathering" (decomposition of the rock) does not affect the rock mass.

In the first class the fracturing degree is extremely high, but not enough to describe the rock mass as crushed and totally fragmented. A blocky-structure is still evident, for this reason the joint volumetric count (J_v) can be evaluated based on the available information about the number of joint sets and the spacing of discontinuities.

According to ISRM recommendations, small (10 joints/m^3 < J_v < 30 joints/m^3) to very small (J_v > 30 joints/m^3) blocks have been considered, therefore, J_v for the first class has been assumed as:

$10 < J_v\ [joints/m^3\ ] < 35$

As mentioned before, if J_v is greater than 35 joints/m^3, the rock mass can be considered as totally crushed.

In the GSI chart for jointed rock masses, the "very blocky" and "blocky/disturbed" groups could be considered as reference point to describe the first class of the highly fractured/faulted zones classification. However, this is done only in terms of fracturing and blockiness, thus without any faulting and folding processes. According to these observations GSI could vary between 15 and 60, depending on the conditions of the discontinuity surfaces.

Being the weathering degree of the rock negligible, the quality of the joint surfaces is supposed no inferior to fair, falling into conditions of moderate alteration. Based on these considerations, the variation range for the GSI narrows and the first class could be defined by the following limits:

30 < GSI < 60

In this class the tectonisation degree is not taken into account, therefore, there is no reference to the "fault" rocks with respect to the definitions and terminology reported in the literature.

In order to characterise the class from a geomechanical point of view, the identification of the most significant parameters becomes necessary. If the fracturing degree and the slightly weathered conditions of the joint surfaces (and rock) are sufficiently described by parameters such as the J_v and GSI, the uniaxial compressive strength of the rock mass UCS_{rm} [77] could be a valid characteristic to consider, because it takes into account the blockiness of the mass and the weathering degree of the discontinuity surfaces. UCS_{rm} is expressed by the following formulation (Eq. 3.10a):

$$UCS_{rm} = s(UCS)^a$$

where UCS is the uniaxial compressive strength of the intact rock, s and a are parameters depending on the geological strength index of the rock mass.

This class describes types of rock mass frequently encountered in tunnelling which can cause stability problems and can often lead to an increasing requirement of supporting measures.

The main geological hazards linked to highly fractured rocks in tunnelling are related to squeezing phenomena, possible instability of the excavation face, frequent blocks falls and large water inflows due to greater permeability of the rock mass because of the extremely high fracturing degree.

5.2.2 II class: highly fractured weathered rock masses

The second class consists of highly fractured rocks with more significant weathering conditions of the rock and of the discontinuity surfaces. The weathering degree may largely influence the geomechanical behaviour of the rock mass and affect its response to excavation.

The rocks have undergone a certain deterioration (in particular along joints) and for this reason they can be treated as "weathered rocks". There are no signs of material decomposition, but a significant discolouration can represent the early stage of "chemical weathering".

Regarding the fracturing degree the same considerations of the ones related to the first class have been developed: a blocky-structure characterises the rock mass and it is still possible to evaluate the joint volumetric count on the basis of the joint sets number and frequency.

The variation range of J_v refers to small and very small blocks, consequently, the upper and lower limits are the same of the ones considered in the first class:

$$10 < J_v \ [joints/m^3] < 35$$

The main difference between the first class and the second one is the geological strength index adopted for the rock mass. In the GSI chart the "very blocky" and "blocky/disturbed" groups are considered as reference points in the same way, but worst conditions of the discontinuity surfaces are taken into account. Referring to the "poor" category (highly weathered surfaces) brings down significantly the GSI value that varies now within the following limits:

$$30 < GSI < 40$$

Although the weathering condition of the rock plays a very important role, the tectonisation degree, resulting from important faulting processes, is not considered and no references to "fault rocks" are reported. The weathering which is taken into account for describing the second class is more related to the influence of specific factors, such as water, on the rock deterioration, without considering alterations in the composition and structure of the rock that are results of significant tectonic activities.

The geomechanical characterisation of the rock masses in the second class refers to three main parameters: J_v and GSI for describing the blockiness and the weathering conditions of joints and rock, and UCS_{rm} for estimating the rock mass behaviour.

The most common tunnelling problems related to this class are more or less the same encountered in the first class. However, the altered conditions of the joint surfaces could strongly affect the rock mass strength and stand-up time (e.g. ravelling).

5.2.3 III class: cohesive fault rocks (and heterogeneous rock masses)

The third class consists of faulted and tectonised rocks. This class considers all zones where the rock behaviour depends on the intensity of the tectonic faulting and where cataclasis is the main degradation process. As a matter of fact the geomechanical properties have undergone a significant alteration due to the tectonic faulting which gradually transforms the rock from the original intact state to a completely crushed soil-like material [8, 76]. Nevertheless, only the early stages of this process have been taken into account in the third class and the “crushed” (non-cohesive) state has not been considered.

As written before, the products of cataclasis (cataclastic/fault rocks) have undergone mechanical weathering, involving brittle fracturing processes, and chemical weathering, favoured by potential migrating fluids. The cohesive cataclastic rocks considered in the third class are characterised by original rock fragments (clasts) in a fine-grained matrix [75]. This chaotic structure results in a complete lack of blockiness and referring to joint sets may become no more appropriate.

Based on the previous observations, the fracturing degree (i.e. J_v) is no longer a reference parameter describing the third class. On the contrary, the weathering (tectonisation) degree remains the main factor for describing this type of rock masses. In order to take into account this aspect, the new versions of the GSI system have been considered, as they have been developed for better representing sheared weak rocks and heterogeneous rock masses.

In particular, this class is partly represented by the “foliated/laminated/sheared” category in the GSI chart for jointed rock mass (Figure 5.1). As already mentioned, this last category has been introduced for describing rock masses with downgraded strength and enhanced deformability, resulting from significant weathering of the intact rock and intense shearing and mylonitisation processes along the lamination or foliation planes [72]. At the same time, this class can be described by some categories in the GSI chart developed for heterogeneous rock masses [73, 74]. In particular, the types of rock mass that show a certain deformation after tectonic disturbance, such as Type VII, Type VIII, Type IX and Type XI (Figure 6.2), are considered. Regarding the quality of lamination/foliation surfaces, only fair to very poor conditions are supposed, given the pre-sheared nature of the rock’s discontinuities [72].

The variation range for GSI in the third class becomes:

$5 < GSI < 40$

Some examples of fault rocks that can be included in this class are generally known in the literature as “cataclasite” or “mylonite” [8, 46, 49, 75, 76], depending if a non-foliated or foliated structure is considered.

For what concerns the geomechanical behaviour of the third class, the uniaxial compressive strength of the rock mass (UCS_{rm}) cannot be any more considered since the fracturing degree and blockiness of rock (expressed by J_v) are no more taken into account. Thus, the uniaxial compressive strength proposed for the cataclastic rocks (UCS_H) by Habimana [8, 76], becomes a valid parameter in order to better characterise the rock mass from the geotechnical point of view. As a matter of fact, UCS_H strongly depends on the tectonisation degree quantified by GSI (Eq. 5.2) and it is expressed the following formulation (Eq. 3.11a):

$$UCS_H = (GSI/100)^{0.55}UCS$$

Regarding the main problems of TBM tunnelling, instabilities in front of the cutterhead, squeezing of the cataclastic material, potential high water inflows, and consequent increase of the pressure on the support represent the most common hazards in rock mass described by the third class. The seriousness of the problem is a function of the weathering of the rock material, for example, tectonisation effect can be reduced by high degree of cementation caused by migrant fluids (“cemented breccia”).

5.2.4 IV class: crushed fault rocks

The tectonically crushed fault rocks and disintegrated rock masses are considered in a separate class. This condition corresponds to the highest degree of tectonisation which entails a soil-like behaviour of the material that can be characterised by a non-cohesive nature. The degradation reaches its peak, from both a mechanical and chemical point of view. Consequently, the geomechanical properties undergo an inevitable worsening because of the weathering of the intact rock material.

The extremely high fracturing degree and no more evident discontinuity sets result in a total absence of blockiness. As for the third class, the joint volumetric count (J_v) cannot be evaluated, but, according to the description given in section 5.2.1, totally crushed rock masses can be supposed for J_v greater than 35 joints/m^3. It is important to underline that this value is only an assumption because there is no enough information for an accurate estimation.

In this context, the term “tectonisation” refers to a significant mechanical fragmentation of the rock and to an important alteration/degradation of the rock material. As a matter of fact, the weathering conditions, expressed by the decreased rock surface quality, still represent an important factor for describing the fourth class where the rock masses experienced very intense shearing and faulting processes.

This class can be represented by the poorly interlocked and heavily broken rocks included in the “disintegrated” category of the GSI chart (Figure 5.1). In the new version of GSI, developed for heterogeneous rock masses (Figure 5.2), the fourth class corresponds to Type IX rock mass, in which it is specified that such fragmented rocks can be found in wide and highly weathered fault zones. With regard to the surface conditions, the quality is supposed to vary from “poor” to “very poor”.

By combining the previous observations, the variation range of GSI narrows and the upper limit becomes the lowest one among all the classes of proposed classification:

$$5 < GSI < 30$$

The fourth class can be associated to particular categories of fault rocks reported in the literature, specifically “fault breccia” and “fault gouge”. They are both described as non-cohesive (or slightly cohesive) material, where the main difference depends on the percentage of visible fragments in the rock mass [46, 60, 75].

As written by Habimana et al. [8, 76], the geomechanical characterisation of heavily tectonically crushed material is not an easy task because knowledge about its mechanical behaviour is still insufficient. Thus again the UCS_H probably remains the most appropriate parameter in order to describe the rock mass in this class from a geotechnical point of view. Based on Eq. 3.11a, the strength of rock can undergo a reduction of between 50% and 80% of UCS, depending on the GSI value.

Regarding the tunnelling problems related to the fourth class, an important role is played by the lithology of the host rock. According to Sausgruber and Brandner [60], the "fault breccia" results from the crushing of competent rocks (high compressive strength and high Young's modulus) giving rise to cohesionless material subject to sudden collapses and characterised by high permeability that entails a significant risk of huge water inflows. On the contrary the "fault gouge" is a slightly cohesive material originated from brittle deformation of incompetent host rocks (low compressive strength and low Young's modulus), characterised by a lower permeability (dripping water at tunnel face) and by a higher squeezing potential.

5.2.5 "Fault zone" classification procedure

Figure 5.4 summarises the procedure adopted for identifying the classes according to the description given in the previous sections.

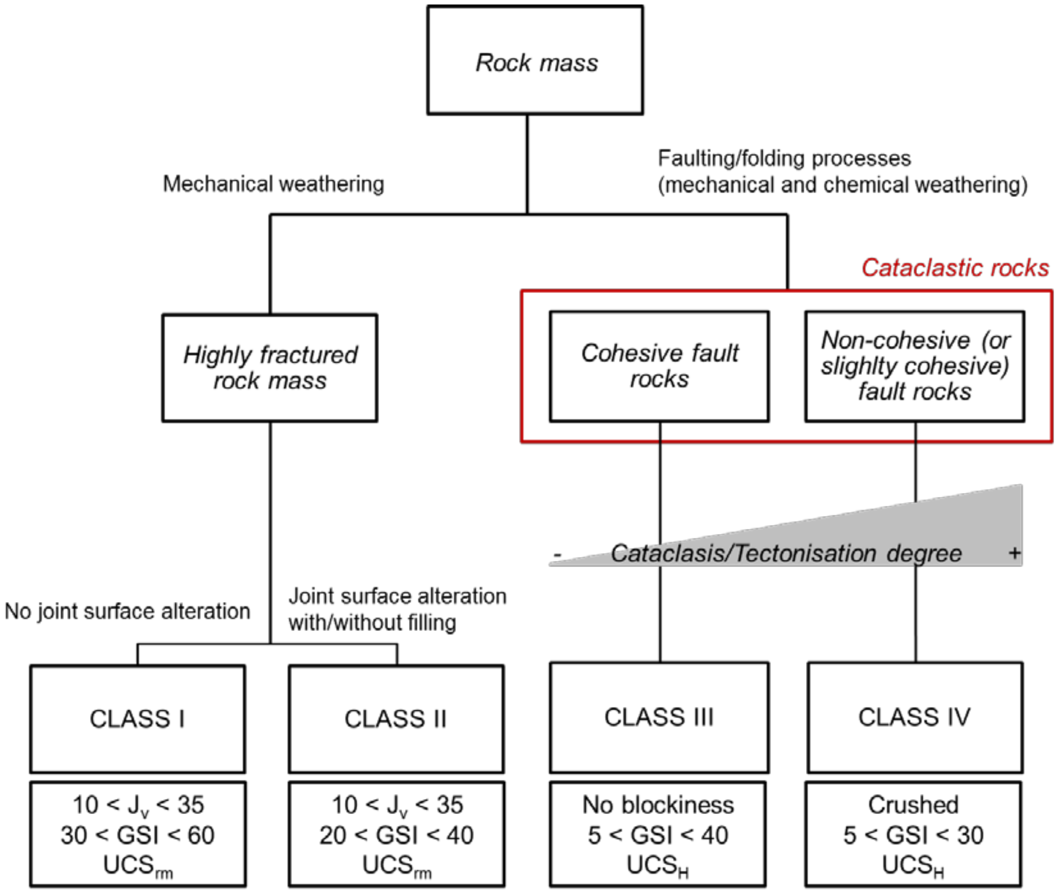


***Fig. 5.4** The procedure adopted for defining the four "fault zone" classes.*

5.3 TBM-performance database in good tunnelling conditions

Though it is quite difficult to evaluate with exactitude the TBM behaviour in a fault zone, the performance decrease of the machine in difficult ground conditions has been studied with respect to the best performance recorded in good tunnelling situations (normal conditions). In order to have information about performance of the TBM in "normal conditions", a new database has been developed. This database contains the same groups of parameters already considered in the database developed for the highly fractured and faulted rock masses (see Chapter 4):

1) TBM specifications and TBM performance (RPM, thrust force, p, PR, daily AR);
2) geological-geotechnical characteristics of the rock mass (rock strength, water inflow, fracturing degree, weathering degree of rock and joint surfaces).

In order to compile the data, attention has been focused on stretches where a total advance rate greater than 15 m/day has been recorded. Generally, these sections are characterised by unweathered rock masses with low or medium fracturing degree and insignificant water inflows. In particular, the following characteristics are met:

- joint volumetric index (J_v) ≤ 20 joints/m^3: massive to slightly fractured/fractured rock mass (see Table 4.6, Chapter 4), where the size of blocks varies from very large to medium, according to the description proposed by ISRM [87];
- weathering degree almost negligible: the rock mass is described as unweathered or slightly weathered and the rock, defined as very strong or strong (UCS > 60 MPa), is characterised by a "good" or "fair" geomechanical behaviour (see Table 4.7, Chapter 4);
- low water inflows: if present, the amount of water does not exceed 60 l/min.

For each tunnel project, a number of sections approximately equal to the number included in the TBM-performance database of highly fractured and faulted rocks has been considered.

The decrease of the machine performance, with respect to the good tunnelling conditions, has been assessed for each lithotype already included in the TBM-performance database developed for highly fractured and faulted rocks. This allowed evaluating the behaviour of the machine in the same lithotype with changing ground conditions. Table 5.1 shows the average values of daily advance rate (m/day) and penetration rate (m/hr, mm/rev) recorded for different lithotypes. Since these values refer to several tunnel projects, each lithotype actually represents a "category" which can be slightly different from project to project. The lithotype "gneiss", for instance, might correspond to crystalline gneiss, gneiss with subordinate micaschist or gneiss with minor amounts of quartzite, calk-silicates and/or pegmatite. Moreover, also the minimum and maximum best performance (i.e. the two highest values recorded for each lithotype during construction) have been reported.

***Table 5.1** Average daily AR [m/day], PR [m/hr] and p [mm/rev] for different lithotype classes with good tunnelling conditions*

Main lithotype	Average daily AR [m/day]		Average PR [m/hr]	Average p [mm/rev]
	Average			
	Min best performance	Max best performance		
Gneiss	22		1.7	5.3
	24	34		
Schist	17		2.6	7.4
	13	28		
Limestone	19		2.2	6.2
	23	26		
Granodiorite	29		2.3	6.3
	28	37		
Greenstone	24		-	-
	29	35		
Amphibolite	30		1.8	5.7
	31	32		

As it is possible to observe, due to the different data quality of the analysed tunnel projects, some information (such as the rate of penetration for greenstone) is not available.

5.4 TBM performance reduction analyses

By having information on the TBM performance in good rock masses, it is possible to estimate the decrease of performance due to difficult ground conditions described by the "fault zone" classes introduced in the previous sections. It is important to underline that the "fault zone" classes are not equally distributed within the TBM-performance database. The first and the second class (highly fractured rock masses characterised by different degrees of weathering) represent the most common conditions encountered in the analysed tunnel projects (Figure 5.5).

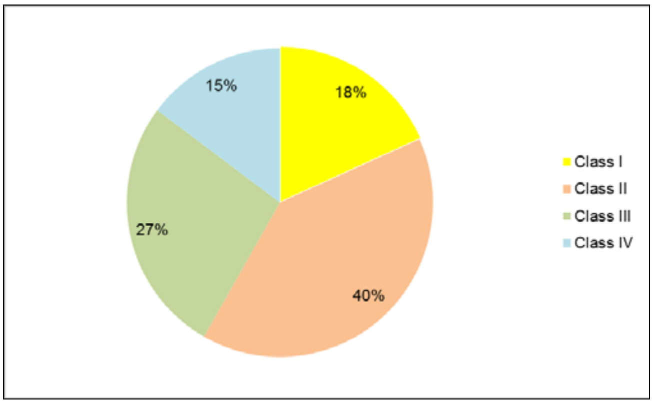


***Fig. 5.5** Percentages of tunnel sections representing the different "fault zone" classes in the TBM-performance database.*

For each class, the TBM-performance parameters, i.e. the penetration per revolution p (mm/rev), the cutterhead rotation speed RPM (rev/min), the penetration rate PR (m/hr) and the daily advance rate AR (m/day), have been "normalised" considering the average values of the best performance (as expressed in Table 5.1).

Unfortunately, while the computation of the daily advance rate has been possible for all tunnel sections included in the TBM-performance database, the values of p, RPM and PR are not available for each tunnel project. Table 5.2 shows the "representativeness" of each "fault zone" class per type of TBM parameter analysed. It is possible to observe that the analyses performed can be always considered representative for more than 50% of the data recorded per each class.

***Table 5.2** Percentages of tunnel sections considered in the estimation of the TBM-performance parameters (for each "fault zone" class)*

TBM parameter	Class 1	Class 2	Class 3	Class 4
p, RPM, PR	76 %	52 %	61 %	71 %
Daily AR	100 %	100 %	100 %	100 %

5.4.1 Penetration per revolution, p

In Figure 5.6 the decrease of the penetration per revolution (p), with respect to the best performance (p_{max}) recorded in good tunnelling conditions, is represented for each "fault zone" class.

In the first "fault zone" class, i.e. highly fractured rock masses (Figure 5.6a), most of the tunnel sections (more than 60% of sections) are characterised by a penetration per revolution approximately equal to the best performance recorded in good tunnelling conditions (p ≥ 80% of p_{max}). A reduction of about 40% to 60% is observed for only 10% of tunnel sections. This result seems to confirm the theory, introduced in the previous chapters, that the high number of joint sets generally does not represent a hindrance to the boring process.

A similar result should be expected also for the second class (Figure 5.6b) which consists of highly fractured and weathered rock masses. However, in this case, more variable values for p have been recorded. As it is possible to observe, about 20% of the tunnel sections are affected by an important performance decrease, down to 20% of the best performance value. Unlike the first “fault zone” class, only approximately 30% of sections exceed the 80% of the best performance recorded in good tunnelling conditions. The lower values of p can be explained by the decreased surface quality of the joints (poor and very poor), where the high weathering degree can results in soft clay fillings. This can represent a problem for the interaction between rock mass and TBM cutters because the excavation may not proceed anymore via the usual chipping process.

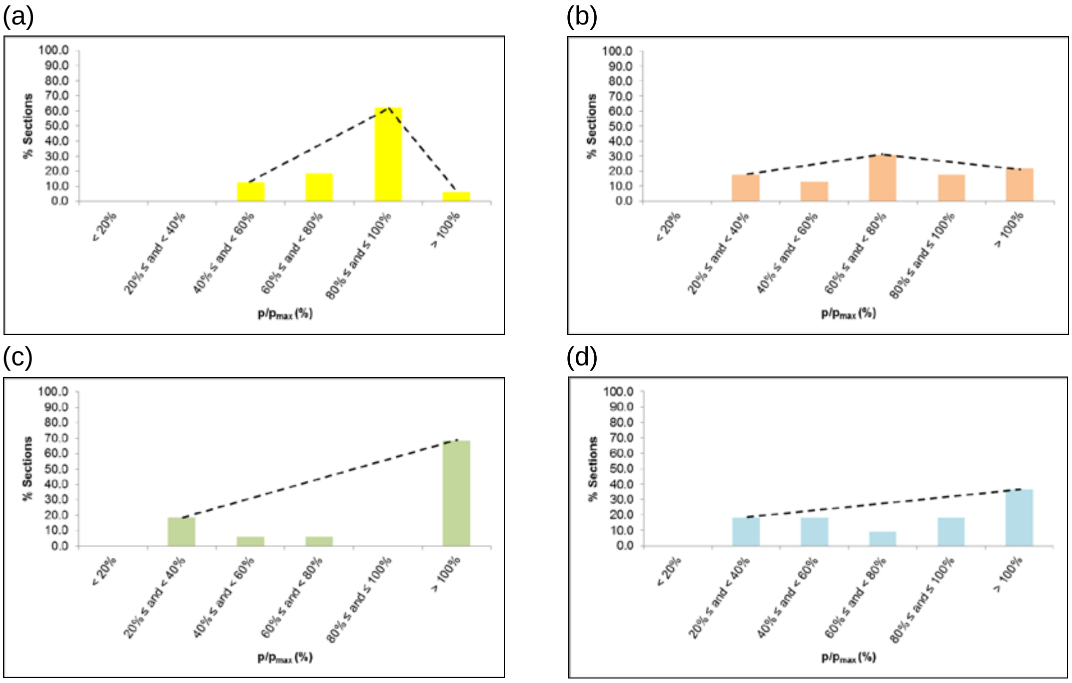


Fig. 5.6 *The reduction of the penetration per revolution p (mm/rev) with relation to the best performance p_{max} recorded in the same lithotypes with good ground conditions. The broken line represents the average trend in each class. (a) Class I; (b) Class II; (c) Class III; (d) Class IV.*

Regarding the third class (i.e. cohesive faulted rocks and heterogeneous rock masses), a totally different behaviour has been observed for the penetration per revolution (Figure 5.6c). Most of tunnel sections (more than 70% of sections) are characterised by a penetration even greater (≥ 100% p_{max}) than the best performance recorded in good conditions. In this class, as already reported in paragraph 5.2.3, the fracturing degree is no longer a reference parameter for describing this type of rock mass. Therefore, though it is not possible to justify the increasing of the penetration with a high number of joint sets, the apparently easier boreability of the rock can be explained by a significant strength reduction due to an intense faulting and folding process.

High values of penetration per revolution have been recorded also for the crushed rock masses described by the fourth “fault zone” class (Figure 5.6d). As it is possible to observe, about 50% of the tunnel sections are characterised by p ≥ 80% of p_{max}, while only about 20% undergoes a halving of the best performance. In this class the rock is characterised by the highest degree of mechanical and chemical weathering, reaching a soil-like state. This almost total loss of strength, on one hand, seems to lead to a better boreability of the rock mass but, on the other hand, the crushed material may reduce the chipping efficiency.

The results of this analysis clearly show that the penetration of the rock by the cutters does not represent a particular issue in highly fractured and fault zones. Thus, the recorded values are not extremely low since a significant fracturing degree (together with the decrease of the rock strength) generally assists the crushing of the material at the tunnel face.

5.4.2 Cutterhead rotation speed, RPM

The cutterhead rotation speed strongly depends on the dimension of the machine, therefore, to do a comparison among several different tunnel projects (with different diameters) can prove to be irrelevant. For this reason, it has been decided to take into consideration the percentage of reduction with respect to the possible maximum RPM of the TBM (RPM_{max}).

In Figure 5.7 the decrease of RPM, with respect to RPM_{max}, is represented for each “fault zone” class.

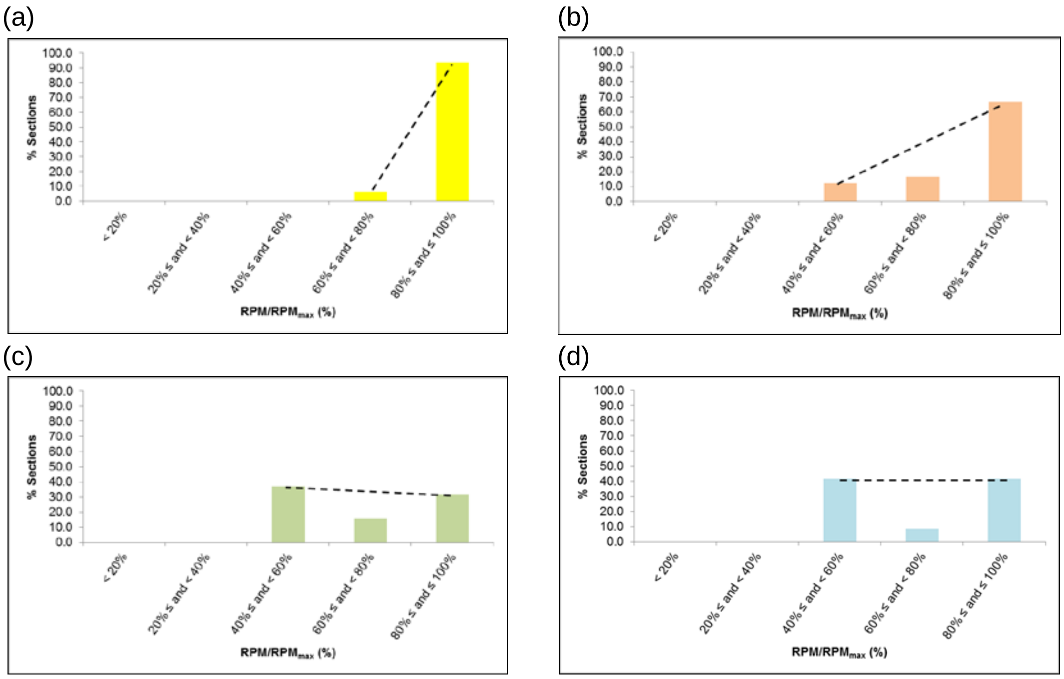


Fig. 5.7 *The reduction of the cutterhead rotation speed RPM (rev/min) with relation to the RPM_{max} of the machine. The broken line represents the average trend in each class. (a) Class I; (b) Class II; (c) Class III; (d) Class IV.*

In the first class (Figure 5.7a) the RPM keeps constant for most of the sections where a minor reduction has been recorded. As a matter of fact, more than 80% of sections are characterised by RPM ≥ 80% of the RPM_{max}, while less than 10% undergo a halving with respect to the maximum value.

A similar behaviour has been observed in the second “fault zone” class (Figure 5.7b), where the high weathering degree of the joint surfaces seems not affecting the RPM of the TBM. However, a certain reduction with respect to the first class has been recorded: less than 70% of sections are characterised by a RPM ≥ 80% of the RPM_{max}, while more than 10% show significant decrease with respect to the maximum value (down to 40% of p_{max}).

In the third class, i.e. faulted rocks (Figure 5.7c) the reduction of RPM is more evident. Approximately 70% of sections are indeed characterised by a RPM ≤ 80% of RPM_{max} and approximately 40% of them undergo a halving with respect to the maximum value.

An important reduction has been observed also in the fourth class, described by crushed rock masses (Figure 5.7d). In this case about 40% of tunnel sections keep the RPM close to the maximum value, while a same number of sections are characterised by RPM between 40% and 60% of RPM_{max}.

The results obtained for the cutterhead rotation speed clearly show the influence of highly fractured and faulted rocks on the TBM performance, especially in the last two “fault zone” classes, which represent indeed the worst tunnelling conditions. As already mentioned in Chapter 4, the RPM undergoes an inevitable reduction in challenging environments (such as crushed rock masses) that hinder operational issues related, for instance, to the efficiency of the muck removal system. Contrary to the effects that bad ground conditions have on the penetration per revolution p (cutter-scale), the reduction of the cutterhead rotation speed RPM more affects the final advancement of the machine (see next sections) because it reflects the excavation problems at the TBM-scale.

5.4.3 Penetration rate, PR

The penetration rate (PR), expressed as meters per hour, represents the distance excavated by the TBM in a continuous excavation phase. This parameter can be considered as a combination between the penetration per revolution (p) and the cutterhead rotation speed (RPM) by the equation 3.2b (Chapter 3):

$$PR\ [m/hr] = p \times RPM \times 60/1000$$

If RPM increases (or decreases), PR can significantly increase (or decrease) aside from the value of p. In other words, PR does not depend only by the real penetration at the cutter scale (mm/rev). Being strongly related to the rotational speed of the cutterhead, it somehow represents the penetration at the TBM-scale (without considering downtimes and possible stops of the machine).

In Figure 5.8 the decrease of PR, with respect to the best performance (PR_{max}) recorded in good tunnelling conditions, is represented for each “fault zone” class.

In highly fractured rocks (first “fault zone” class) most of the tunnel sections (about 80%) are characterised by PR ≥ 60% of PR_{max} (Figure 5.8a). For more than 50% of them, PR ≥ 80% of PR_{max} has been recorded, while about 10% of sections undergo a halving of the best performance. The same considerations which are already made for the penetration per revolution (p) can be applied in this case: the penetration of the rock does not represent the biggest issue in highly fractured rocks and good performance can be achieved also in these difficult ground situations. Moreover, RPM shows a negligible reduction in this class, keeping a value close to RPM_{max} for the majority of the sections. The high values of p are therefore confirmed by the high values of PR.

In the second class more variable values of p have been recorded and the same variability has been observed for the rate of penetration PR (Figure 5.8b). Also in this case a low percentage of tunnel sections (about 20%) exceed the 80% of the best performance achieved in good tunnelling conditions. However, a greater reduction has been recorded for PR, with respect to the reduction observed for p. As a matter of fact, more than 30% of sections show a significant decrease and about 10% of them reach values lower than 20% of PR_{max}. This reduction of PR (even if not so evident) compared to p, is due to the reduction recorded for the cutterhead rotation speed RPM.

In faulted rocks (the third class) the tunnel sections are characterised by rather high PR (Figure 5.8c). The reduction of RPM observed in this class affects the percentage of sections interested by the highest values of PR. For about 30% of sections, values between 80% and 100% of PR_{max} have been recorded, while for the penetration per revolution p the values even exceed 100% for more than 60% of sections.

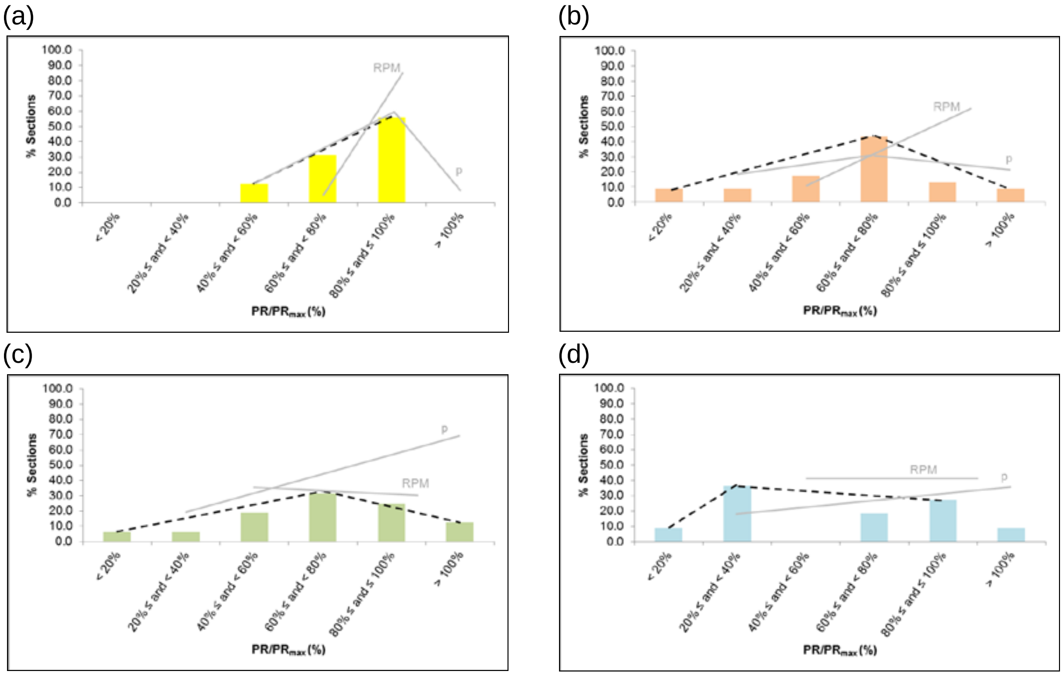


***Fig. 5.8** The reduction of the penetration rate PR (m/hr) with relation to the best performance PR_{max} recorded in the same lithotype in good ground conditions. The broken line represents the average trend of PR in each class. The grey lines represent the average trend of p and RPM. (a) Class I; (b) Class II; (c) Class III; (d) Class IV.*

In the fourth class the values of PR do not reflect what has been observed for p (Figure 5.8d). Despite approximately 30% of tunnel sections are characterised by PR ≥ 80% of PR_{max}, for more than 40% of sections PR undergoes a strong decrease with respect to the best performance recorded in good conditions, down to values < 20% of PR_{max}. In addition, the evident reduction of RPM in the fourth class seems to affect more PR than in the third class. As a matter of fact, it has been observed that in crushed rocks a greater percentage of tunnel sections are affected by the significant decrease (40% to 60% of RPM_{max}) of the cutterhead rotation speed compared to faulted rocks and heterogeneous rock masses.

5.4.4 Daily advance rate, Daily AR

The daily advance rate (daily AR) is expressed as meters per day and represents the real advancement of the TBM. Since it takes into account also downtimes and stops of the machine, it is probably the performance parameter more affected by uncertainty. Though by using time planning of construction site it has been possible to fitter out holiday and major maintenance time, the advance rate remains a function of the utilisation factor (U) of the TBM. As previously discussed, the quantification of U in very bad ground conditions can become impossible because of the influence of countless factors such as the geological conditions, the performance of the equipment and the human component. However, mainly due to its direct link with PR (see Equation 4.3), a certain trend can be defined for the daily AR in each “fault zone” class.

Figure 5.9 shows the decrease of daily AR, with respect to the daily AR_{max} (the average value between the two best performance recorded in good tunnelling conditions), for each “fault zone” class.

As reported in Table 5.2, the daily AR has been estimated considering all tunnel sections included in the database. Therefore, it is characterised by a greater reliability compared to the penetration per revolution p, the cutterhead rotation speed RPM and the penetration rate PR. In order to better describe the performance reduction in each class and to reach a higher degree of detail, smaller intervals have been considered.

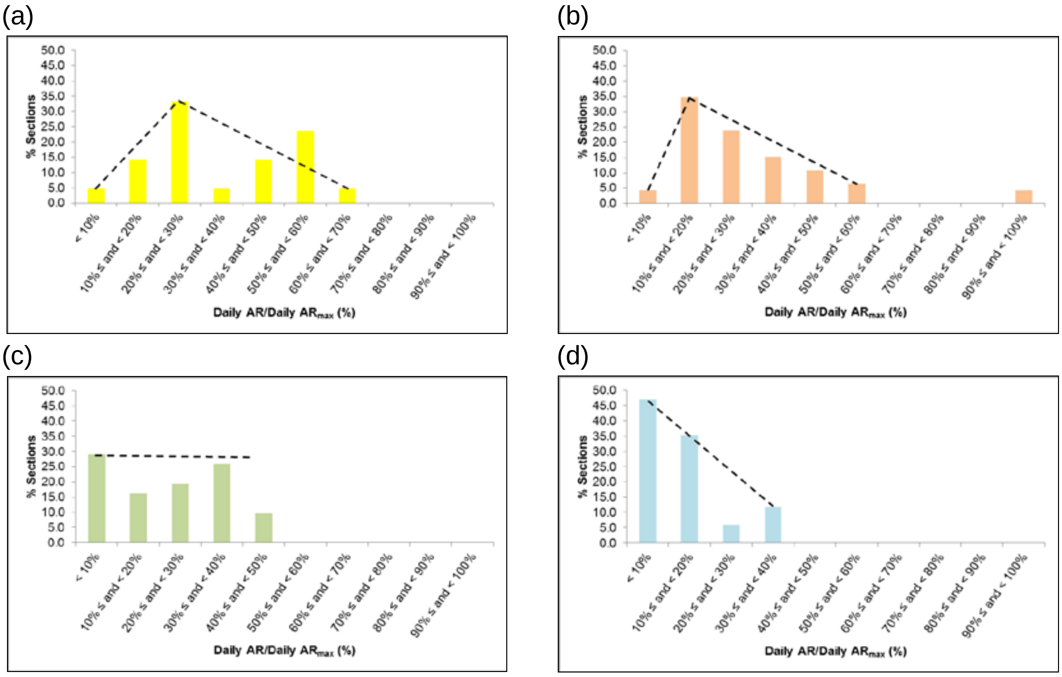


***Fig. 5.9** The reduction of the daily advance rate AR (m/hr) with relation to the best performance daily AR_{max} recorded in the same lithotypes in good ground conditions. The broken line represents the average trend of daily AR in each class. (a) Class I; (b) Class II; (c) Class III; (d) Class IV.*

In the first “fault zone” class (Figure 5.9a) about 30% of tunnel sections are characterised by a daily AR comprised between 20% and 30% of the best performance daily AR_{max} while more than 10% of sections undergo a significant reduction, down to values lower than 20%. Daily AR greater than 50% of daily AR_{max} is recorded for approximately 30% of sections, but they do not exceed 70% of the best performance.

The second “fault zone” class shows a different trend (Figure 5.9b), where more than 60% of tunnel sections are characterised by a highly reduced daily AR (lower than 30% of daily AR_{max}). About 40% of sections reach percentages lower than 20% of the best performance, while about 20% have daily AR comprised between 40% and 60% of daily AR_{max}. There is a very low percentage (about 5%) of tunnel sections where the daily AR exceeds the 90% of the best performance recorded in good conditions. However, this percentage is so low and so distant from the mean value of the class, that it has not been considered for defining the average trend of the daily AR.

In the third “fault zone” class (faulted rocks), the reduction of the performance is still evident but differently spread with respect to the first two classes (Figure 5.9c). A specific trend is not visible because the percentages are quite evenly distributed among the tunnel sections. The daily AR does not exceed 50% of the best performance and more than 60% of sections are characterised by values lower than 30% of daily AR_{max}. The lowest values (lower than 10% of the best performance) have been recorded for about 30% of sections.

The most significant reduction has been recorded in the last class (crushed rocks) where the daily AR proves to be lower than 40% of daily AR_{max} in each tunnel section (Figure 5.9d). However, unlike the faulted rocks, the vast majority of the sections are characterised by values lower than 10% of the best performance (more than 45% of sections). About 90% of tunnel sections show values that do not exceed 30% of daily AR_{max}.

5.5 Comments and recommendations

The results of the analyses performed in this chapter allow to confirm what was previously discussed about TBM penetration rate. As a matter of fact, the TBM rate of penetration (expressed as p or PR), does not seem to be the major issue in highly fractured and faulted rock masses. However, concerning PR, a certain performance decrease is more evident due to the influence of the cutterhead rotation speed (RPM) of the machine. This is certainly due to the different scale of the problem. At the tool-scale (i.e. p) the face stability issues do not represent a hindrance to cope with, while at the tunnel diameter scale (i.e. RPM) the size of the TBM is introduced, thus also all the issues related to the face instability.

The analyses show that the values of the penetration per revolution p, are similar to the values recorded in good tunnelling conditions. A significant performance reduction has not been found, quite the contrary, a certain increase has been even observed in the last two “fault zone” classes (Figure 5.10). As already mentioned several times, the high fracturing degree, together with the strength reduction undergone by the rock material (especially for the third and fourth classes), seem to facilitate the boring process rather than to hinder it.

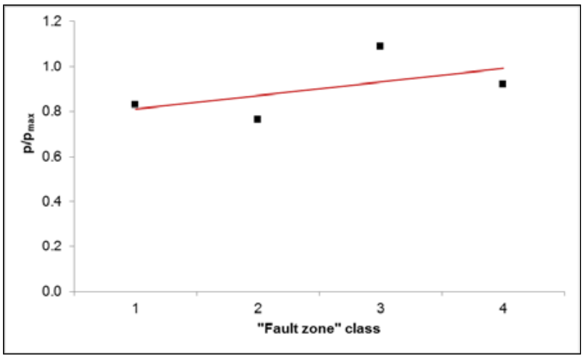


Fig. 5.10 *The average reduction of the penetration per revolution (p) in each “fault zone” class, with respect to the best performance recorded in good tunnelling conditions (p_{max}).*

The RPM seems to be more affected by the worsening of the ground conditions. As a matter of fact, a certain decrease can be observed in the last two classes (which correspond to the worst tunnelling conditions), while in the first and second classes the values are very close to the maximum RPM of the machine (Figure 5.11). However, it is important to remind that RPM is also affected by the skill of the TBM operator, as well as by the possibility to adapt the rotation speed according to the changing ground conditions right on time.

The RPM reduction has an important influence on the penetration rate PR of the TBM, for which a decreasing trend has been observed (Figure 5.12).

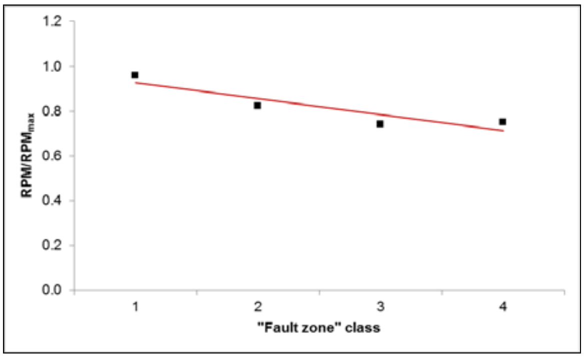


Fig. 5.11 *The average reduction of the cutterhead rotation speed RPM in each “fault zone” class, with respect to the maximum RPM of the TBM (RPM_{max}).*

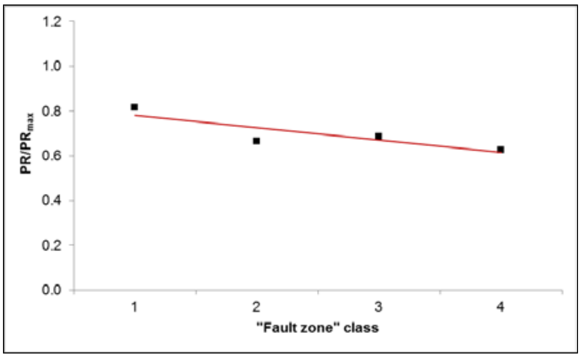


Fig. 5.12 *The average reduction of the rate of penetration PR in each "fault zone" class, with respect to the best performance recorded in good tunnelling conditions (PR_{max}).*

The daily advance rate is the parameter which undergoes the greater reduction in highly fractured and faulted rock masses, with respect to the performance recorded in good tunnelling conditions. A certain decreasing trend has been observed from the first to the fourth "fault zone" class (Figure 5.13). According to what is shown in Figure 5.13, a quite important reduction of the daily AR (down to 40% of the best performance) can be observed with just an increasing degree of fracturing characterising the first fault class. Then, the difference between the reduction percentages becomes smaller from one class to the next one. The minimum is clearly reached with the fourth class (representing the worst excavation conditions) where the majority of tunnel sections are characterised by daily AR lower than 10% of the best reference performance (see Figure 5.9d).

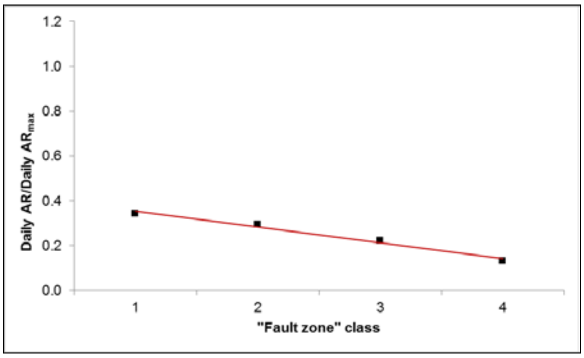


Fig. 5.13 *The average reduction of the daily advance rate (daily AR) in each "fault zone" class, with respect to the best performance recorded in good tunnelling conditions (daily AR_{max}).*

However, it is important to underline again, that the advance rate of the TBM strongly depends on the utilisation factor U of the machine which is affected by countless factors. As already mentioned, it is extremely difficult to establish a direct correlation between U and the geological/geotechnical conditions of the rock mass, consequently, to exactly predict the advance rate is practically impossible [44]. The variable human factors such as the experience of the crew, the contractual decisions, the working conditions, etc. can be estimated only taking into account a certain uncertainty.

In conclusion, referring to the developed "fault zone" classification and to the results derived from the TBM performance analysis carried out for each class, the following important observations can be pointed out:

1. In order to characterise highly fractured and fault zones, it is necessary to take into account the mechanical and chemical weathering undergone by the rock mass. The weathering degree affects the engineering properties of the material; therefore, specific geological and geotechnical parameters are adopted in this framework to obtain a more effective description and characterisation of the behaviour of the rock mass.
2. Wider ranges of parameters generally used for characterising rock mass should be taken into account: the new categories of GSI (better representing the altered state of

the rock subject to folding and faulting processes) are considered together with significant fracturing degrees (expressed by high J_v, up to values greater than 30 joints/m^3). Based on this information, the reduced strength of the rock UCS_H (strongly dependent on the tectonisation degree of the material by the GSI) is introduced.

3. On the basis of the estimation of these geological and geotechnical parameters, four classes of rock mass are identified, in this study, as different groups describing a “fault zone” state.
4. In order to estimate the TBM performance in these difficult environments, the analysed tunnel sections are described taking into account the geological and geomechanical parameters considered in the proposed “fault zone” classification.
5. Once each section is associated to a specific “fault zone” class, it is possible to evaluate the TBM performance according to the reduction rates applied to the best performance. The best performance can be estimated in good tunnelling conditions by means of one of the TBM-performance prediction models reported in the literature.
6. For each “fault zone” class a specific reduction of TBM performance has been identified and pointed out. This reduction does not concern the cutter-penetration (mm/rev) in the rock (which even increases with the fracturing degree and with the decrease of the rock strength), but it refers to the penetration rate PR (m/hr) of the TBM (affected by the reduction of the cutterhead rotation speed RPM) and to the final advancement of the machine (daily advance rate AR expressed in m/day).

The reduction rates applicable to the TBM-performance parameters in each class can be summarised in Table 5.3. In order to better differentiate the performance decrease in the four classes, also the “mode range” of reduction evaluated for each of them is reported.

Table 5.3 *The rates of reduction, with respect to the best predicted TBM performance, to be applied to the TBM-performance parameters (for each “fault zone” class)*

TBM parameter	**Rate of reduction (to be applied to the best performance: RPM_{max}, PR_{max}, Daily AR_{max})**			
	Class I	**Class II**	**Class III**	**Class IV**
RPM [mm/rev]	80% - 100%	60% - 100%	40% - 100%	40% - 100%
	Mode range	***Mode range***	***Mode range***	***Mode range***
	80% - 100%	*80% - 100%*	*40% - 60%*	*40% - 60%*
PR [m/hr]	40% - 100%	20% - 100%	20% - 100%	20% - 100%
	Mode range	***Mode range***	***Mode range***	***Mode range***
	80% - 100%	*60% - 80%*	*60% - 80%*	*20% - 40%*
Daily AR [m/day]	10% - 70%	10% - 60%	10% - 50%	10% - 40%
	Mode range	***Mode range***	***Mode range***	***Mode range***
	20% - 30%	*10% - 20%*	*< 10%*	*< 10%*

It is important to underline that the percentages reported in Table 5.3 strongly depend on the database developed for this study. The category of tunnel project, the TBM type, the experience of the crew, the contractual liabilities and the quality of the recorded data represent only some factors affecting the obtained results. The database can be surely improved by increasing the number of projects with the aim to apply the proposed approach to different case-studies.

However, although their limitations, the outcomes of this analysis may provide useful insights and they give a first qualitative overview on the reduction of the TBM performance in difficult ground conditions such as highly jointed and fault zones.

6 Probabilistic analysis of TBM tunnelling in highly fractured and faulted rocks with the Decision Aids for Tunnelling (DAT)

The results achieved in the previous chapter, regarding the reduction of the TBM performance characterising each “fault zone” class, have been used to run probabilistic analyses of the tunnel construction. The aim is obtaining time and cost estimation in highly fractured and faulted rocks. These analyses have been performed with the “Decision Aids for Tunnelling” (DAT). The DAT are a model and a computer code that can be used to evaluate the effect on tunnel construction cost and time of the uncertainties related to both the geological/geotechnical conditions and the construction process [92-94].

In this section, the DAT have been used to determine the risks related to tunnelling in highly fractured and faulted rock masses. In particular, by taking into consideration different ground conditions, i.e. fault zone classification presented in Chapter 5, the construction of a 1 km long tunnel has been simulated to quantitatively assessing the effects that degrading ground conditions may have on TBM tunnelling in terms of construction time and costs.

6.1 The Decision Aids for Tunnelling

Tunnel construction is always characterised by a high degree of uncertainty, which is linked to two principal aspects [44]:

1) the geological conditions along the tunnel alignment, which are never exactly known;
2) the construction process, which includes the variable performance of crew and equipment, repairs and maintenance occurring at various intervals, as well as accidents/breakages.

All this leads to the practical impossibility to exactly predict advance rates and costs.

The DAT can formally quantify uncertainties in tunnelling thus providing a basis to determine risk. This is done by simulating the actual construction of a tunnel (or a network of many tunnels) in a number of different possible geological profiles. Most of the parameters governing the simulation can be defined in a probabilistic way, resulting in the overall uncertainty about a project with respect to construction cost and time [92]. The DAT have been applied in a number of cases [95, 96] and have been further developed to consider details of resource modelling [97] and applications to linear/networked infrastructure projects involving structures other than tunnels [98]. A recent application [99] uses the DAT to model muck removal and reuse. Finally, the DAT have been also used to compare TBM excavation versus drill & blast excavation in difficult ground conditions [100].

The DAT are a toolbox consisting of two main modules, called “geology module” (description of geology) and “construction module” (simulation of construction). The geology module produces probabilistic geologic/geotechnical profiles that indicate the probabilities of particular geological conditions occurring at a particular tunnel location. They are usually obtained through a combination of objective information and subjective estimates by experts. A third module, called “resource module”, allows one to consider production and consumption of resources (construction material, muck material, labour or equipment) and to model the effect of interfering activities.

Following a hierarchical structure, the geology along a tunnel is initially subdivided into “zones”. The term “zone” is used to identify a stretch of ground which may be described as a “geologically homogeneous zone” characterised by a particular set of parameters, and their parameter state distribution. Each of these zones consists of a set of "segments" in which a specific set of parameter states occur [101]. The hierarchy of the geological subdivision is represented in Figure 6.1.

Usually, Markov chains are used to simulate the segment sequence of the parameter states along the zones. The Markov process is characterised by mean lengths of the states and transition probabilities from each state to the others [44]. The mean lengths and the transition probabilities of the parameter states are generally subjectively estimated. With reference to Figure 6.1, this is equivalent to define the mean lengths of the states "gneiss", "schist" and "granitic gneiss" and the probability with which the gneiss will change to either schist or granitic gneiss. After computing possible Markov chains for each parameter, these are combined together into a possible ground class profile along each zone. A number of simulations are then performed to obtain several possible ground class profiles along the tunnel alignment to represent the whole range of geological conditions (Figure 6.1 shows for example the result of a single geology simulation). Alternatively, a semi-deterministic or fully-deterministic approach may be used to generate the segment lengths.

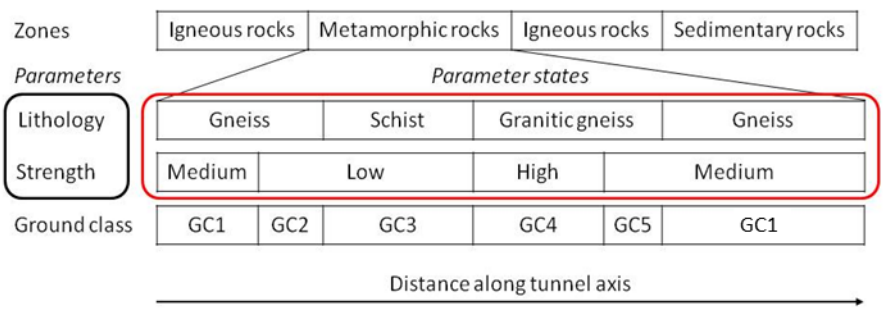


Fig. 6.1 *Hierarchical representation of the geological input in the DAT.*

After running the geology simulations, the construction module simulates tunnel construction through each of the ground class profiles produced by the geology module. Specifically, each ground class is related to a particular "construction method" which defines the excavation procedure and the support requirements [101]. Each method is then associated to construction costs and time, which are generally defined in the form of cost per linear unit of tunnel and advance rate. The construction methods may be represented as a sequence of different activities (boring, supporting, mucking, etc.), each of them defined by a time and cost equation. Triangular distributions are generally used for the input definition. In addition, uniform and lognormal distributions may also be used.

The construction of the tunnel is simulated by advancing round by round through one possible geological profile and a final cost and final time value is produced. If many simulations are run, a so-called time-cost "scattergram" is produced (Figure 6.2).

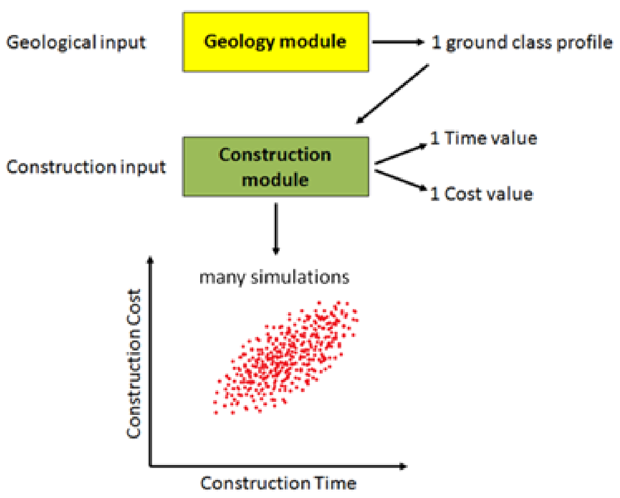


Fig. 6.2 *Schematic diagram of the DAT methodology (after [100]).*

6.2 Effects of highly fractured and faulted rocks on TBM tunnelling simulations with DAT

As discussed in the previous chapters, TBM operational issues related to bad ground conditions represent a primary source of construction delays and cost increase.

In Chapter 5, the reduction of the TBM performance related to degrading tunnelling conditions has been investigated. In order better investigate the influence that faulted and highly fractured rocks have on tunnel construction time and costs, DAT models have been built. According to the DAT methodology, the input definition is subdivided into two main parts: the geology module and the construction module. They are described separately in the following sections.

6.2.1 Geology module

The first step in the development of a DAT model is the definition of the geology input. The construction of a circular, 1 km long tunnel has been considered (this stretch of tunnel can be considered as a part of a longer one). The tunnel lies in a single zone, while the different segments along the alignment, i.e. the ground classes, are defined according to the fault zone classification described in Chapter 5. In particular, the ground parameter “ROCK MASS CONDITIONS” may assume the following four values/states:

1) good tunnelling conditions, C_0 (corresponding to the TBM best performance);
2) highly fractured rocks, C_1 (corresponding to class 1 of the classification);
3) highly fractured, weathered rocks C_2 (corresponding to class 2 of the classification);
4) faulted rocks, C_3 (corresponding to class 3 of the classification);
5) crushed rocks, C_4 (corresponding to class 4 of the classification).

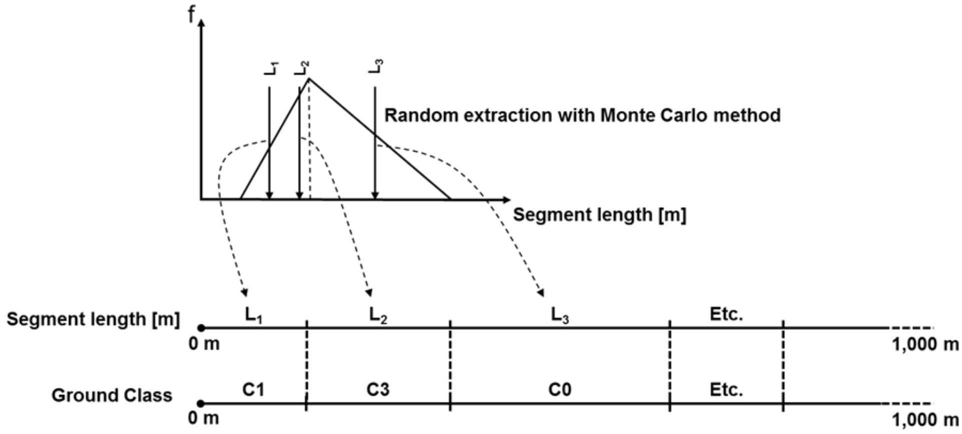
The segments sequence along the alignment is simulated with a Markov process. Different ground conditions are simulated by changing the relative percentage of each class along the tunnel (i.e. via the transition matrix). More in detail, the following three models are considered:

- **Model 1**: quite good geological conditions are supposed and the percentages presented in the first row of Table 6.1 are considered. 70% of class C_0 corresponds, for instance, to 700 m excavated in good rock mass conditions. For this model only 5% of the tunnel is excavated in very bad ground conditions (classes C_3 and C_4) and the rest (10%) is excavated in C_2.
- **Model 2**: The most degrading tunnelling conditions, C_3 and C_4 classes, cover 10% of the tunnel alignment. While class C_2 keeps the same percentage.
- **Model 3**: a worst-case scenario is considered for more than 50% of the alignment, in particular, classes C_3 and C_4 reach a maximum length of about 200 m.

Table 6.1 *Ground class definition along the tunnel alignment*

Model n.	C_0	C_1	C_2	C_3	C_4
1	70 %	15 %	10 %	3 %	2 %
2	60 %	20 %	10 %	5 %	5 %
3	40 %	25 %	15 %	10 %	10 %

An example of segment generation along the tunnel is presented in Figure 6.3.



Transition matrix

GC (from \ to)	C_0	C_1	C_2	C_3	C_4
C_0	-	0.50	0.20	0.20	0.10
C_1	0.50	-	0.30	0.10	0.10
C_2	0.20	0.40	-	0.30	0.10
C_3	0.20	0.20	0.30	-	0.30
C_4	0.10	0.20	0.30	0.40	-

Segment lengths

GC [MPa]	Segment lengths [m]		
	Min	Mode	Max
C_0	150	200	250
C_1	30	40	50
C_2	20	30	40
C_3	6	12	18
C_4	6	12	18

Fig. 6.3 *Example of segment sequence along the alignment for Model 1. The probability to go from a given class to another one is defined by the transition matrix, while the length of each segment is picked with a Monte Carlo extraction from the triangular distribution defined by the table "segment lengths".*

6.2.2 Construction module

Tunnel construction is simulated with the DAT construction module. This allows combining the geological information with the construction process. In particular, for each combination of tunnel geometry (i.e. in this case only one geometry, corresponding to a gripper TBM, is considered) and ground class, a construction method is defined. Each construction method may be considered as a sequence of tunnelling activities, defined in terms of time and cost equations. The variables used in the equations are called "method variables" (as their Probability Density Function, PDF, varies from one method to another) and the combination of all the defined activities is called "activity network" [101].

In the present case, and five different construction methods have been defined. Each of this method corresponds to one of the five ground classes defined in the geology module, as described here below:

- **Method 1**: TBM excavation in good tunnelling conditions, corresponding to class C_0.
- **Method 2**: TBM excavation in highly fractured rocks, corresponding to class C_1.
- **Method 3**: TBM excavation in highly fractured and weathered rocks, corresponding to class C_2.
- **Method 4**: TBM excavation in faulted rocks, corresponding to class C_3.
- **Method 5**: TBM excavation in crushed rocks, corresponding to class C_4.

Each method is based on a single activity. In the present case, a single construction activity ("TBM advance") representing the total TBM advance rate, thus including TBM boring and the production delays (e.g. support installation, machine maintenance, need for ground improvements, etc.) is considered. This is actually a common procedure for defining the construction module.

In the DAT, activities are defined in terms of time and cost equations (generally expressed as a function of the advance rate and cost per meter). Therefore, the results derived from the analysis of the TBM performance data presented in the previous chapter are used.

The best TBM performance in good tunnelling conditions defined as daily AR_{max} (in this chapter the daily AR_{max} is called AR_0 for simplicity) are considered. For this distribution (AR_0) the data retrieved from the database developed for the good conditions are used (Figure 6.4).

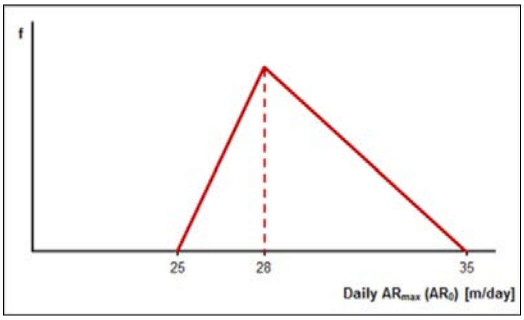


Fig. 6.4 *Triangular distribution function for* AR_0.

Unfortunately for what concerns excavation costs there is no information available. Therefore for the modelling the final costs have been defined by taking into account data taken from literature. Figure 6.5 show the triangular distribution considered for representing the excavation cost per meter in good tunnelling conditions (EC_0). This distribution is representative of the excavation cost of a hard rock machine with a diameter of about 8 to 12 m (i.e. the same type of gripper TBM included in the TBM performance database).

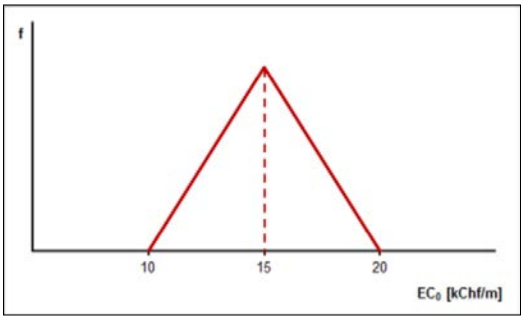


Fig. 6.5 *Triangular distribution function for the excavation cost* EC_0.

In order to take into account the TBM advance rate and cost per meter reduction due to degrading ground conditions along the tunnel alignment the performance reduction identified in Chapter 5 for each fault zone class have been used.

The excavation time t_i (where i= 1 to 4 refers to each ground class) for each method, is thus given by the following equation:

$$t_i = RL/(AR_0 \cdot r_i) \quad (6.1)$$

where RL is the round length (set equal to 2.0 m, i.e. one TBM stroke), AR_0 is the best performance advance rate and r_i is the reduction factor (probabilistic) computed for each ground class on the basis of the associated histogram plot (see Chapter 5).

The term r_i is introduced in the DAT as triangular distribution for C_1, C_2, C_4 (Figures 6.6a, 6.6b and 6.6d) and as uniform distribution for C_3 (Figure 6.6c). As a matter of fact, while in the first, second and fourth classes it has been possible to define a mode value, in the third class only a maximum and a minimum advance rate have been identified (see Figure 5.10c). The uniform distribution function seemed therefore the best choice (i.e. same probability to peak any value comprised between the minimum and the maximum). Regarding the distribution adopted for C_4, the mode value has been assumed equal to the minimum value (0.1). This is due to the fact that the vast majority of the tunnel sections described by this class are characterised by advance rate lower than 10% of the best performance (see Figure 5.10d).

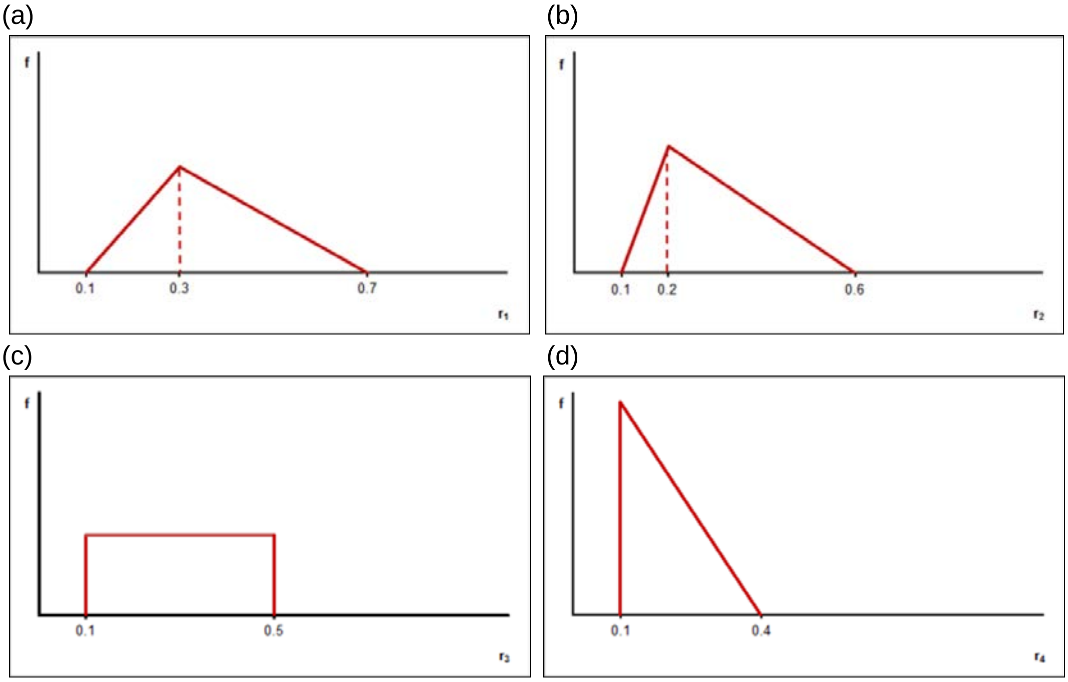


Fig. 6.6 *Distributions of the term r_i (i=1,2,3,4) adopted for each rock mass class. The minimum value, the maximum value and the mode value are put in evidence. (a) C_1; (b) C_2; (c) C_3; (d) C_4.*

If the TBM advance rate reduces according to Equation 6.1 as the ground conditions get worse, the excavation costs increase accordingly. In particular it has been assumed that the construction costs increase proportionally to $1/r_i$. (Equation 6.2):

$$c_i = RL \cdot EC_0 \cdot (1/r_i) \quad (6.2)$$

According to Equation 6.2, a significant cost increase is calculated as one moves from class C_0 to class C_4.

6.3 Simulation results

For the three models 10 simulations of the geology have been run, each of them produces a different ground class profile. Then, for each ground class profile, 100 construction simulations have been executed, thus, giving 10 x 100 simulations of tunnel construction. The model results are represented in the form of a time-cost “scattergram”. As already mentioned, each point of the diagram represents the result of a single construction simulation through a possible ground class profile.

The results for models 1, 2 and 3 are represented respectively in Figures 6.7, 6.8 and 6.9. The details of each simulation are summarised in Table 6.2, and the scattergrams of the three models are finally compared also in Figure 6.10.

Table 6.2 and Figure 6.10 show clearly an increase of construction time and costs while changing from good (i.e. model 1) to bad conditions (i.e. model 3). These results are visible by the shifting of the point clouds representing models 2 and 3 towards increasing values of time and costs (see also Figures 6.11a and 6.11b).

Table 6.2 *Details of the DAT output for the three models. St. dev. = standard deviation and C. V. = coefficient of variation (st. dev./mean).*

Model n.	Construction time [day]					Construction cost [millions Chf]				
	Min	**Mean**	**Max**	**St. dev.**	**C. V.**	**Min**	**Mean**	**Max**	**St. dev.**	**C. V.**
1	40.0	60.0	105.0	8.5	0.14	20.0	35.0	62.0	5.0	0.14
2	45.0	70.0	140.0	14.9	0.18	22.5	43.0	75.0	7.1	0.17
3	50.0	85.0	155.0	16.8	0.20	30.0	53.0	87.5	10.4	0.19

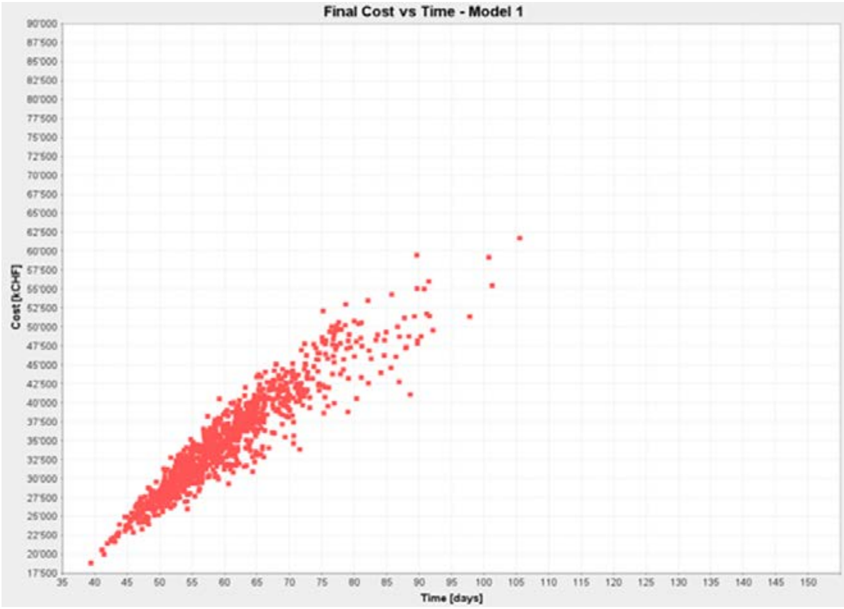


Fig. 6.7 *Time-cost scattergram for model 1.*

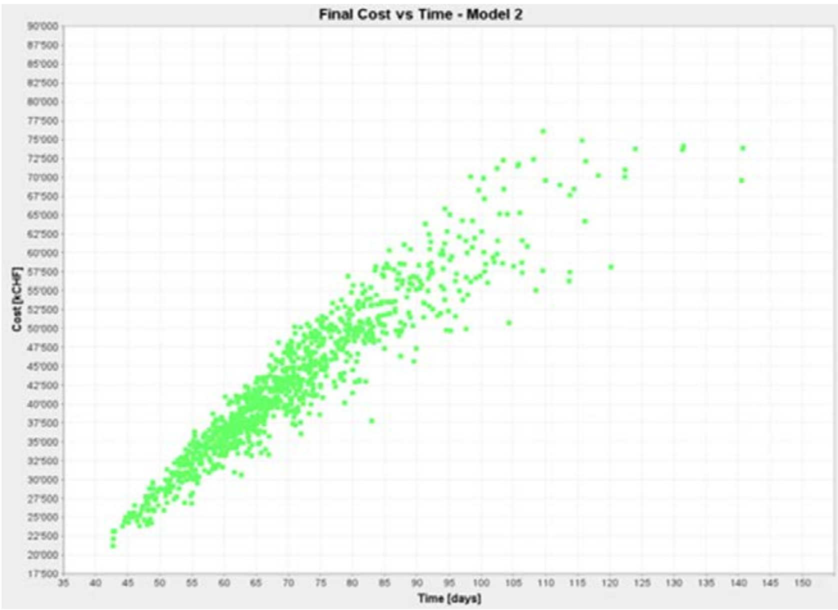


Fig. 6.8 *Time-cost scattergram for model 2.*

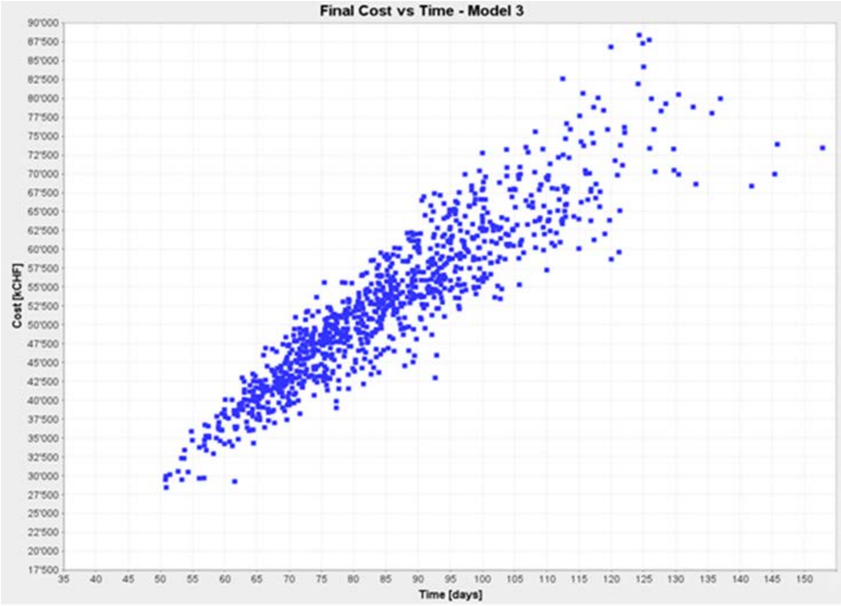


Fig. 6.9 *Time-cost scattergram for model 3.*

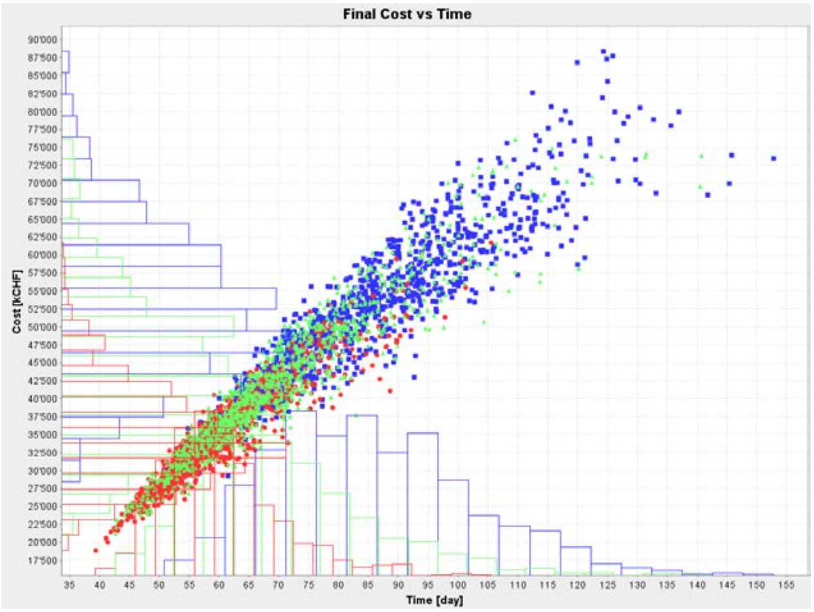


Fig. 6.10 *Time-cost scattergram and histograms for simulations based on three models. Blue cloud: model 1; green cloud: model 2; red cloud: model 3.*

(a) (b)

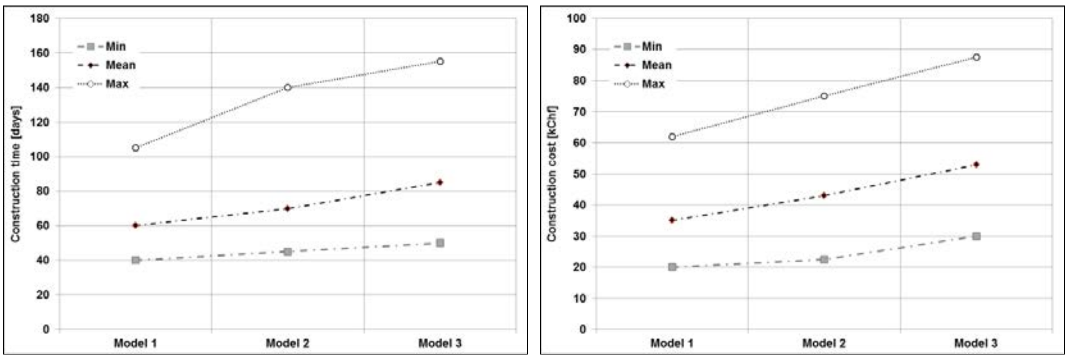


Fig. 6.11 *(a) Construction time for the three considered models; (b) construction cost for the three considered models.*

The results of the analyses show clearly:

- A significant increase of the mean construction time (about 16%) and of the mean construction costs (about 21%) from model 1 to model 2. Moreover, also the cloud scatter (represented by the coefficient of variation C.V.) increases, which represents a higher uncertainty related to degrading tunnelling conditions moving from the first to the second set of simulations.
- An increase of the mean construction time of about 25% and of the mean construction cost of about 41% from model 1 to model 3. Moreover, in model 3 (i.e. worst-case scenario) the C.V. reaches its maximum, which means that in this case, the greatest uncertainty is associated to the estimation of the construction time and costs.
- An increase of the maximum construction time and cost which is much more pronounced with respect to the variation of the mean and minimum values. As a matter of fact, the risk of high/very high construction times (due to machine delays) and cost overruns becomes more significant as the geological conditions get worse.

Although the obtained results are quite interesting to get an idea of the possible delays and cost overruns in highly fractured and faulted rocks, it should be noted that the assumptions made may lead to a final construction costs higher than what can be observed in reality, especially for the worst case (model 3) which represents extreme conditions. This is due to the assumed cost evaluation relationship, expressed by Equation 6.2, which generates very high values as the rock mass conditions get worse.

Finally, it is also interesting to compare the method costs per cycle versus time per cycle for a given model (model 2 is taken as reference but the results are similar for all the models). This analysis represents practically a comparison of the TBM performance in each of the five defined rock mass classes (i.e. Good Conditions, and 4 Fault Zones). The results are reported in Figure 6.12. A significant increase of the construction time and costs in each advancement cycle is observed moving from class C_0 to class C_4.

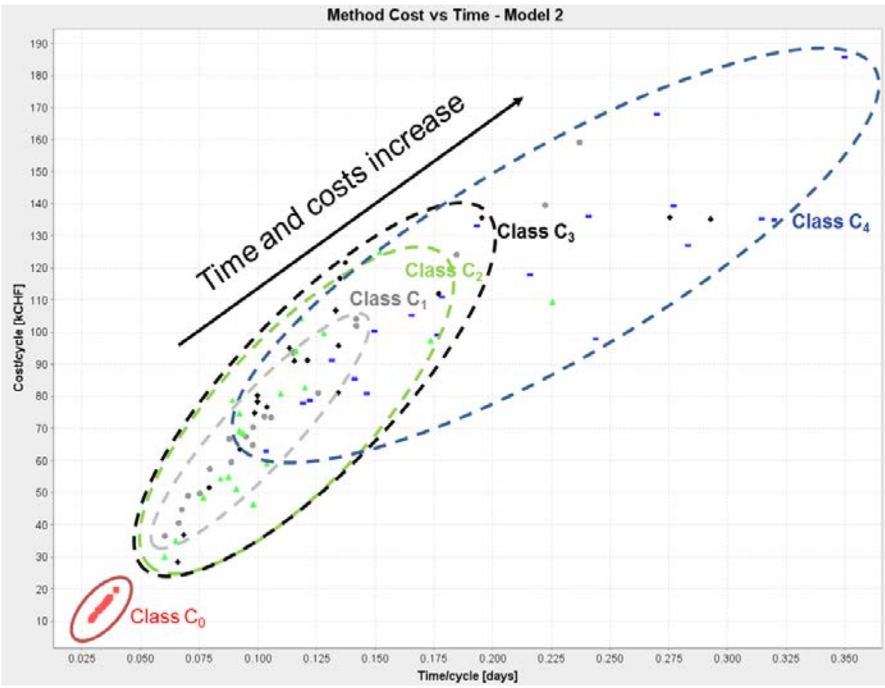


Fig. 6.12 *Method costs vs. time chart for model 2.*

6.4 Final remarks

The Decisions Aids for Tunnelling are a very useful tool for quantifying uncertainties affecting tunnel construction and estimating construction time and costs in variable/uncertain conditions. In this framework, the DAT have been used to estimate the tunnel construction time and costs distributions which may be experienced in highly fractured and faulted rocks.

In particular this analysis allows to more deeply investigate the effect of the identified “fault zone” classes on the tunnel construction. Based on the proposed classification system (see Chapter 5), the following ground conditions have been used as input for the DAT geology module:

- Good, massive to slightly jointed rock (class C_0)
- Highly fractured rock (class C_1)
- Highly fractured and weathered rock (class C_2)
- Faulted rock (class C_3)
- Crushed rock (class C_4)

With different length for each ground type, the DAT models allow considering different rock mass behaviours ranging from good (i.e. with a small percentage of weak zones) to bad conditions, in which faulted and crushed rock are encountered on a considerable percentage of the whole tunnel alignment.

The construction of the tunnel has been simulated by taking into account the results obtained from the analyses performed on the TBM performance data compiled in the TBM performance databases (see Chapter 5). In particular, for class C_0 the best performance recorded in good ground conditions has been considered, while for class C_1, C_2, C_3 and C_4 the rates of reduction obtained for each “fault zone” class have been used to estimate the best performance. Finally, an assumption of increasing costs with increasing excavation times has also been made.

The results of the simulations show how highly fractured and faulted rocks can influence tunnel construction conditions. As a matter of facts, a significant increase of the construction time and costs occurs while changing from “good” to “bad” ground conditions. Furthermore, also a greater uncertainty of simulation results is observed with degrading rock mass conditions, underlining the increasing difficulty in estimating the tunnelling performance.

These results confirm the importance of what has been observed also in the previous chapters (i.e. TBM-performance reduction associated to a specific “fault zone” class), in addition, they allow obtaining a more reliable estimation and evaluation of the final tunnel construction time and costs, considering different possible ground scenarios.

7 Summary and conclusions

The mechanical excavation by Tunnel Boring Machines (TBMs) is nowadays more and more considered a valuable alternative to conventional technology and the application in mining and tunnelling engineering has increased significantly in recent years. In good rocks, generally, the great advantages of gripper TBMs can be summarised as follows: continuous operation, safe working conditions, reduced damage of the surrounding ground, and high advance rate. At the same time, TBM operations may be seriously affected by difficult ground conditions, such as highly fractured and fault zones, with a consequent significant reduction of the TBM performance with respected to expected ones (i.e. good rock masses).

“Fault zone” is commonly used to identify brittle shear zones generated in the upper part of the Earth’s crust ranging from decimetres to kilometres in magnitude. Their extreme complex structure and heterogeneous nature, due to combined contribution of weak and strong rock components, make their geotechnical characterisation very difficult. For example, rocks resulting from brittle faulting processes (e.g. cataclastic rocks) can show properties typical of hard rocks to soil-like materials. This is mainly depending on the character, cohesive or non-cohesive, in relation to the variable ratio of gouge matrix to rock blocks. Therefore, weak rocks resulting from brittle processes are characterised by quite low mechanical strength thus showing a behaviour somewhere between rocks and soils. They also show high deformability involving non-linear constitutive laws, strong dependence of the strength on water saturation and temperature, and great susceptibility to weathering.

The most common geotechnical problems resulting from tunnelling in these kind of rocks include instability of the excavation face, instabilities of side-walls (also producing large overbreaks of the tunnel profile), excessive deformations and strongly anisotropic wall displacements, frequent changes of stresses in terms of magnitude and direction, high water inflows, etc. Thus, a significant reduction of the TBM advance rate (e.g. in worst cases a stop, jamming, of the machine can be observed) is generally recorded, also due construction delays associated to the need of ground treatment, heavier supports and additional waterproofing measures. Furthermore, the presence of a faulted/heavily jointed tunnel face might strongly affect the interaction between rock mass and TBM cutters since the excavation may not proceed anymore via the usual chipping process, consequently causing unexpected wearing of the disc tools and more frequent maintenance of the cutterhead.

In these last decades many TBM performance prediction models have been developed in order to estimate the optimum TBM advance rates for standard (expected as good) ground conditions. Due to the fact that in highly fractured rocks and fault zones many events can take place at the same time (e.g. major high pressure inflows, outwash of fines, large tunnel deformations and formation of cavities at the tunnel crown or ahead of TBM), those models, however, are not representative of bad conditions where it is generally suggested to use them with great caution.

TBM-PERFORMANCE DATABASE AND PRELIMINARY ANALYSES. In order to better analyse the TBM performance in difficult grounds and to investigate the relationships existing between machine performance parameters and geological/geotechnical parameters of the rock mass, a TBM-performance database has been created. The sections included in the database have been selected from different tunnel projects where highly jointed rock masses and/or fault zones have been encountered. The database is mainly subdivided into two sections: the first one contains the tunnel characteristics, TBM specifications and TBM performance parameters (penetration rate, advance rate, etc.); the second one considers the geological-geotechnical parameters of the rock mass (rock strength, fracturing and weathering degree of joints, etc.).

Concerning the data collection, the calculation of the basic machine parameters has been generally performed on the basis of the measures carried out by the TBM on-board acquisition system during construction. However, it was not possible to gain complete TBM data for all tunnel projects included in the database. As a matter of fact, the contractors are often unable to share easily the data from the site especially when on-going claims have not been resolved yet. Regarding the compilation of the geological and geotechnical parameters, it is important to underline that they have been directly gathered from the tunnel projects or estimated from laboratory tests through formula commonly used in rock mechanics. Although the degree of detail of data varies from one project to another, enough information has been collected in order to have an idea of the distribution of important parameters related to the boring process and the TBM advancement.

In order to identify the correlations between the performance parameters of the machine and the geological/geotechnical characteristics, preliminary analyses have been carried out based on the information collected in the TBM-performance database. The first results confirm the difficulties in selecting, within existing prediction models, the parameters affecting the most the TBM performance while degrading the ground conditions. As a matter of fact, in highly fractured rocks and fault zones, the delays associated to the increasing need for heavy temporary supports, rock jams, gripper bearing failure, additional drainage system, etc., are partially balanced by the rather high TBM penetration rates generally observed in fractured/very fractured rock masses. Moreover, find a correlation between the TBM advancement and the rock mass quality is a quite difficult task because of the huge number of parameters, both rock mass- and construction- related, involved in its definition as well as the unpredictable events, often due to the human factor which is almost impossible to quantify (TBM utilisation factor). In order to partially solve this problem, the reduced uniaxial compressive strength [8, 76], which takes into account the weathering degree of rock mass, has been introduced. Despite a significant lowering of strength values (thus more representative of the real conditions), the scattering of the results remains and the correlation with the TBM performance parameters is almost meaningless. All of these uncertainties can be considered one of the major causes of the large data scattering found in these preliminary analyses. Thus, in order to consider the rock mass conditions as a whole, i.e. as a combination of factors, it has been decided to investigate the relationships existing between the TBM performance parameters (penetration rate and advance rate) and the Rock Mass Rating. The results have been consistent to previous works made by Sapigni et al. [35]. Though some trends could have been identified, the RMR system seemed to be still not sufficient for characterising the behaviour of difficult ground conditions zones and thus obtaining a better prediction of TBM performance. As a matter of fact, it has been observed that some parameters which are basic factors for the evaluation of the RMR, (e.g. the rock strength) and extremely significant to estimate hard rock tunnel boring, may actually play a very marginal role in the definition/estimation of the machine performance in highly fractured and faulted zones.

FAULT ZONE CLASSIFICATION AND CORRELATIONS WITH THE TBM PERFORMANCE. In order to consider the complex structure of fault zones and bearing in mind that a more global approach is required for characterising their behaviour (i.e. combination of factors instead of specific parameters approach), the next step has been the development of a classification system of these particular environments. In order to identify the specific classes describing highly fractured and faulted rocks, several geological/geotechnical parameters have been taken into account. The choice of specific parameters has been done by compiling several literature sources dealing with fault's characterisation both from geological and geotechnical point of view. In particular the fracturing degree of the rock mass (expressed by the joint volumetric index, J_v) and the weathering degree of the rock and of the joint surfaces (expressed by the Geological Strength Index, GSI) have represented the main factors describing the characteristics and the behaviour of each "fault zone" group. Four classes have been defined:

1. *Highly fractured rock mass*: the rock mass is characterised by a high fracturing degree ($10 < J_v$ [joints/m^3] < 35); there are no visible signs of discolouration and decomposition of the rock material and/or of the joint surfaces. This class can be described by the Uniaxial Compressive Strength of the rock mass (UCS_{rm}), depending on UCS of the intact rock and on GSI ($30 < GSI < 60$).

2. *Highly fractured weathered rock mass*: in addition to the high fracturing degree ($10 < J_v$ [joints/m^3] < 35), the weathering conditions of the rock and/or of the joint surfaces get worse but there are still no signs of alteration in composition and structure of the rock material (no results of faulting activities). This class can be still described by UCS_{rm}, but the range of GSI varies with respect to the first class ($20 < GSI < 40$).
3. *Cohesive fault rocks (and heterogeneous rock masses)*: the fracturing degree (i.e. J_v) is no longer a reference parameter and the weathering degree remains the main factor for describing this class and it represents the results of faulting and folding activities (tectonisation). The uniaxial compressive strength proposed by Habimana [8, 76] for the cataclastic rocks (UCS_H) becomes a valid parameter in order to better characterise the rock mass from the geomechanical point of view. The UCS_H depends on GSI ($5 < GSI < 40$) that varies according to the degree of tectonisation undergone by the rock material.
4. *Crushed fault rocks*: disintegrated rock mass that represents the highest degree of tectonisation and degradation of the material. The extremely high fracturing degree and no more evident discontinuity sets result in a total absence of blockiness (J_v is assumed to be greater than 35 joints/m^3). The UCS_H remains the most appropriate geomechanical parameter in order to describe this class where GSI reaches the lowest values ($5 < GSI < 30$).

Once the “fault zone” classes have been defined, it has been possible to evaluate the TBM performance in each of them, in terms of penetration rate and daily advance rate. A second database has been developed including tunnel sections characterised by good tunnelling conditions. The aim was to analyse the best TBM performance recorded during the excavation and to compare it with the behaviour of the machine in difficult ground conditions (faulted and highly fractured rock masses).

For each “fault zone” class a reduction rate with respect to the best performance has been defined for the following TBM-performance parameter: the cutterhead rotation speed RPM (rev/min), penetration rate PR (m/hr) and daily advance rate AR (m/day). It is important to underline what has been observed for the penetration of the rock (penetration per revolution, p) expressed in mm per revolution. Despite to expectations, a certain increase has been even observed in the last two “fault zone” classes (cohesive and crushed fault rocks) where the high fracturing degree, together with the strength reduction undergone by the rock material, seems to facilitate the boring process rather than to hinder it. Contrary to PR, p somehow refers to the penetration at the tool-scale, where the face stability problems do not represent a hindrance to cope with. On the other hand, with PR, the tunnel diameter scale is introduced because depending on RPM, therefore also the issues related to face instability affect the performance. Regarding the advance rate AR, a decreasing trend has been observed from the first to the fourth “fault zone” (representing the worst tunnelling conditions).

THE “DECISION AIDS FOR TUNNELLING”. The TBM performance reduction (in terms of advancement) observed for each “fault zone” class obviously traduces in potentially longer tunnel construction times and higher costs. In order to more deeply analyse the influence that faulted and highly fractured rocks may have on the final time and costs, a probabilistic analysis of the tunnel construction has been performed with the Decision Aids for tunnelling (DAT). The DAT are computer based tools that allow engineers to simulate the construction keeping into consideration the uncertainties related to geology and to excavation processes. The DAT consist of two main modules describing respectively the geological conditions (“geology module”) and the construction process in terms of methods and sequence (“construction module”). The “geology module” produces probabilistic ground class profiles indicating the probabilities of particular geological conditions occurring at a particular tunnel location. The excavation of the tunnel network through these profiles is then simulated by the construction module. The input is usually obtained through a combination of objective information and subjective estimation done by experts. Based on the proposed classification system, the following ground conditions have been input in the DAT geology module:

1. Good, massive to slightly jointed rock (class C_0, corresponding to good tunnelling conditions)

2. Highly fractured rock (class C_1, corresponding to Class I of the “fault zone” classification)
3. Highly fractured and weathered rock (class C_2, corresponding to Class II of the “fault zone” classification)
4. Faulted rock (class C_3, corresponding to Class III of the “fault zone” classification)
5. Crushed rock (class C_4, corresponding to Class IV of the “fault zone” classification)

The construction of the tunnel has been simulated by taking into account the results of the analyses performed on the data compiled in the TBM-performance databases (i.e. developed both in good and bad conditions). The results of the DAT analysis confirmed the significant construction time (and costs) increase that may take place moving from “good” grounds to severe geological conditions. Thus, according to the different TBM-performance reductions observed in each “fault zone” class, it has been possible to obtain a reliable estimation of the final tunnel construction time and costs, considering different possible ground scenarios. Beside the results achieved in terms of time and costs quantification, it is important to underline how the information obtained from the analysis carried out on the TBM-performance database partly reduced the uncertainties related to the subjective estimation generally made by the DAT user. These uncertainties are higher in difficult ground conditions, such as fault zones, where the machine performance (in terms of advancement) is hard to assess.

To conclude, the results of this study allow obtaining a better evaluation of the TBM behaviour in highly jointed rocks and fault zones, although the limitations due to the quality of the geological/geotechnical information and the problems relating to the accessibility of TBM data. The proposed classification tries to better characterise the “fault zone” environments from a geomechanical point of view, and it allows providing a quantification of the performance reduction in such difficult ground conditions. The effect of the worsening of the TBM performance on the final construction time and cost has been even further assessed by detailed probabilistic simulations.

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Project closure report

Schweizerische Eidgenossenschaft
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Département fédéral de l'environnement, des transports, de l'énergie et de la communication DETEC
Office fédéral des routes OFROU

RECHERCHE DANS LE DOMAINE ROUTIER DU DETEC Version du 09.10.2013

Formulaire N° 3 : Clôture du projet

établi / modifié le : 15.10.2014

Données de base

Projet N° :	FGU 2007/004
Titre du projet :	TBM Tunnelling in Faulted and Folded Rocks / Construction mécanique de tunnels dans des roches faillées et plissées
Echéance effective :	29.04.2013

Textes :

Résumé des résultats du projet :

This project focuses on the study of the performance of Tunnel Boring Machines (in particular gripper TBMs) in highly jointed rock masses and fault zones.
In order to investigate possible relationships between difficult rock mass conditions and TBM performance, the data of several tunnel projects have been collected in a specific database (TBM-performance database). This database compiles data obtained directly from the field, laboratory tests and literature and it is divided into two sections: the first one contains the tunnel characteristics, TBM specifications and TBM performance parameters; while the second one considers the geological-geotechnical parameters of the intact rock and rock mass.
Preliminary analyses have been carried out in order to identify correlations between TBM performance parameters (namely, penetration and advance rates) and rock mass parameters generally used in common TBM performance prediction models, such as rock strength and fracturing degree. Although some trends could be identified, the scattered results confirm the difficulties in predicting the machine performance in complex geological environments starting from existing models and characterising indexes proper to good rocks. Thus, a specific classification system for highly fractured rock masses and fault zones has been developed to cope with these problems. The aim of this classification is to select representative parameters that should be considered in order to obtain a more complete geomechanical description of disturbed zones. Four "fault zone" classes have been identified. Through data analyses, it has been possible to evaluate for each class a specific reduction rate of TBM-performance parameters (i.e. penetration rate, cutterhead rotation speed and daily advance rate) with respect to the best tunnelling performance recorded in good ground conditions. The results of these analyses allow to effectively quantify the effect of degrading rock mass conditions on the TBM advance (in terms of both penetration and daily advance rates). Moreover, these results have been used to carry out probabilistic analyses of the tunnel construction time and costs by means of the "Decision Aids for Tunnelling" (DAT). The outcomes of those simulations give a reliable estimation of the effect of degrading geological conditions (from highly fractured rocks, to faulted rocks, to crushed material) on tunnel excavation.
The results of this research may provide useful insights regarding the reduction of the machine performance that can take place in bad grounds, with respect to an estimated best performance in good tunnelling conditions.

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Schweizerische Eidgenossenschaft
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Office fédéral des routes OFROU

Atteinte des objectifs :

The following objectives have been attained:

- Establishment of a TBM-performance database in both bad and good ground conditions with data of major tunnel projects.
- Evaluation of the effects of highly fractured and faulted rocks on the TBM penetration and advancement by means of statistical analyses.
- Development of a geomechanical classification for fault zones and highly fractured rock masses and identification of four principal classes.
- Definition, for each "fault zone" class, of a reduction rate with respect to the best performance for selected TBM-performance/operational parameters.
- Probabilistic simulations of tunnel excavation in order to evaluate the effects of degrading rock mass conditions on the final tunnel construction time and costs.

Déductions et recommandations :

Based on the outcomes of this study, the following recommendations can be provided:

- If bad ground conditions are expected, it is important to take into account a reduction of TBM performance with respect to an estimated best performance in good tunnelling conditions (Chapter 1).
- The estimation of the TBM performance is affected by a high degree of uncertainty. The existing performance prediction models are no more reliable (Chapters 1 and 4).
- The creation of a specific database compiling information from existing projects allows reducing uncertainties related to tunnelling in difficult ground conditions (Chapter 3).
- The characterisation of highly fractured and fault zones requires the consideration of the mechanical and chemical weathering undergone by the rock mass. It is important to identify the rock degradation degree (i.e. Fault Zone Classification) by taking into account specific geological and geotechnical parameters (Chapter 5).
- The TBM-performance prediction values, described through selected parameters such as penetration rate, cutterhead rotation speed and daily advance rate, obtained for the good tunnelling conditions via the existing prediction models should be corrected by introducing the reduction rate obtained for each class (Chapter 5).
- In order to obtain more reliable estimation of time and costs overruns while crossing high disturbed / fault zones it is important to simulate different combinations of fault zone classes and lengths (Chapter 6).

Publications :

E. Paltrinieri and F. Sandrone (2014). A study of the TBM performance in fault zones and highly fractured rocks, in World Tunnel Congress 2014: Iguassu Falls, Brazil.

E. Paltrinieri, J.P. Dudt, F. Sandrone, and J. Zhao (2014). Comparison of excavation methods in different grouns conditions with the Decision Aids for Tunnelling, in 15th Australasian Tunnelling Conference 2014: Sydney, NSW.

Paltrinieri, F. Sandrone and J. Zhao (2014). Analysis and estimation of the TBM performance in highly fractured and faulted rocks. Journal article, in preparation.

Chef/cheffe de projet :

Nom : Zhao Prénom : Jian

Service, entreprise, institut : EPFL, Laboratoire de mécanique des roches (LMR)

Signature du chef/de la cheffe de projet :

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Département fédéral de l'environnement, des transports, de l'énergie et de la communication DETEC
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RECHERCHE DANS LE DOMAINE ROUTIER DU DETEC

Formulaire N° 3 : Clôture du projet

Appréciation de la commission de suivi :

Evaluation :

Tunnelling in fault zones is explicitly difficult and has thus a significant impact on the overall costs and time. Fault zones show a large variety in geological and hydrogeological characteristics, geometrical dimensions and geotechnical behaviour. Beside this given features also the TBM parameters (torque, thrust etc) together with the handling of the machine and the overall more or less appropriate reaction of the stakeholders involved makes it very difficult to address the TBM tunnelling in fault zones in a scientific manner. There are very many parameters to consider whose interactions and interlinkage are nearly impossible to analyze and to predict. In consequence of this simplified and rather generic approaches are chosen which in most of the cases refer to a smaller or larger number of case studies (the relevance of which for the research at stake is in most of the cases not further evaluated).
Also in this research work this approach has been selected. But as it is stated on many occasions in the report this approach didn't allow to draw significant conclusions, due to various reasons: too small or inadequate data base of the given case studies, difficulties in describing adequately the relevant geological, geotechnical and hydrogelogical parameters, not sufficient information about other influencing parameters which didn't allow to properly extract the relevant parameters, too wide numerical range of various parameters including large areas of overlap etc.
In the end the work may be considered as a valuable contribution to the overall understanding of some of the relevant parameters and it allows some generic insights into the topic. However the conclusions are both too vague and limited due to various reasons and they thus may not be of significant help in the design and construction of TBM tunnelling in fault zones.

Mise en oeuvre :

The base of the work consits in the comparision of different existing formulas for TBM advance rates and their basic geological input data, which later on were applied to the geological characteristics of fault zones. However both the formulas and the geotechnical characteristics of fault zones are quite generic which made it difficult to derive tangible relations.
In order to overcome this the work introduces 4 classes of fault zones and tries to describe them refering to some geotechnical parameters which as such does not always follow pure scientific criteria.
Additionally and partly in the attempt to compensate for the rather small theoretical input, the work is also based on data from some case studies which do in itself differ quite much (type of TBM, size of TBM, rock parameters, overburden etc). This makes it again difficult to extract and compare the needed basic characteristics which might serve as input for the correlations searched.
In the end statistical evaluations with the so called 'Decision aids on Tunnelling' were carried out, which result in rather large areas of cost-time point clouds for the different classes, confirming in a general manner the expected trends but being too vague to allow specific conclusions.

Besoin supplémentaire en matière de recherche :

The topic as such is of high importance but as long as the fundamental approaches remain the same and refer to existing semi- or fully empirical formulas and correlations obtained from case studies it seems not to be reasonable to carry out additional research work as it will not considerably enlarge the fundamental scientific understanding of the relevance of the different paramaters and their interlinkage.

Influence sur les normes :

Existing codes and standards will not be affected by the carried out research work.

Président/Présidente de la commission de suivi :

Nom : Amberg Prénom : Felix

Service, entreprise, institut : Amberg Engineering AG, Trockenloostrasse 21, 8105 Regensdorf-Watt

Signature du président/ de la présidente de la commission de suivi :

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1459	VSS 2002/501	Leichtes Fallgewichtsgerät für die Verdichtungskontrolle von Fundationsschichten	2014
1458	VSS 2010/703	Umsetzung Erhaltungsmanagement für Strassen in Gemeinden - Arbeitshilfen als Anhang zur Norm 640 980	2014
1457	SVI 2012/006	Forschungspaket VeSPA Teilprojekt 5: Medizinische Folgen des Strassenunfallgeschehens	2014
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1416	FGU 2010/001	Sulfatwiderstand von Beton: verbessertes Verfahren basierend auf der Prüfung nach SIA 262/1, Anhang D	2013
1415	VSS 2010/A01	Wissenslücken im Infrastrukturmanagementprozess "Strasse" im Siedlungsgebiet	2013
1414	VSS 2010/201	Passive Sicherheit von Tragkonstruktionen der Strassenausstattung	2013
1413	SVI 2009/003	Güterverkehrsintensive Branchen und Güterverkehrsströme in der Schweiz Forschungspaket UVEK/ASTRA Strategien zum wesensgerechten Einsatz der Verkehrsmittel im Güterverkehr der Schweiz Teilprojekt B1	2013
1412	ASTRA 2010/020	Werkzeug zur aktuellen Gangliniennorm	2013
1411	VSS 2009/902	Verkehrstelematik für die Unterstützung des Verkehrsmanagements in ausserordentlichen Lagen	2013
1410	VSS 2010/202_OBF	Reduktion von Unfallfolgen bei Bränden in Strassentunneln durch Abschnittsbildung	2013
1409	ASTRA 2010/017_OBF	Regelung der Luftströmung in Strassentunneln im Brandfall	2013
1408	VSS 2000/434	Vieillissement thermique des enrobés bitumineux en laboratoire	2012
1407	ASTRA 2006/014	Fusion des indicateurs de sécurité routière : FUSAIN	2012
1406	ASTRA 2004/015	Amélioration du modèle de comportement individuell du Conducteur pour évaluer la sécurité d'un flux de trafic par simulation	2012
1405	ASTRA 2010/009	Potential von Photovoltaik an Schallschutzmassnahmen entlang der Nationalstrassen	2012
1404	VSS 2009/707	Validierung der Kosten-Nutzen-Bewertung von Fahrbahn-Erhaltungsmassnahmen	2012
1403	SVI 2007/018	Vernetzung von HLS- und HVS-Steuerungen	2012
1402	VSS 2008/403	Witterungsbeständigkeit und Durchdrückverhalten von Geokunststoffen	2012
1401	SVI 2006/003	Akzeptanz von Verkehrsmanagementmassnahmen-Vorstudie	2012
1400	VSS 2009/601	Begrünte Stützgitterböschungssysteme	2012
1399	VSS 2011/901	Erhöhung der Verkehrssicherheit durch Incentivierung	2012
1398	ASTRA 2010/019	Environmental Footprint of Heavy Vehicles Phase III: Comparison of Footprint and Heavy Vehicle Fee (LSVA) Criteria	2012

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1397	FGU 2008/003_OBF	Brandschutz im Tunnel: Schutzziele und Brandbemessung Phase 1: Stand der Technik	2012
1396	VSS 1999/128	Einfluss des Umhüllungsgrades der Mineralstoffe auf die mechanischen Eigenschaften von Mischgut	2012
1395	FGU 2009/003	KarstALEA: Wegleitung zur Prognose von karstspezifischen Gefahren im Untertagbau	2012
1394	VSS 2010/102	Grundlagen Betriebskonzepte	2012
1393	VSS 2010/702	Aktualisierung SN 640 907, Kostengrundlage im Erhaltungsmanagement	2012
1392	ASTRA 2008/008_009	FEHRL Institutes WIM Initiative (Fiwi)	2012
1391	ASTRA 2011/003	Leitbild ITS-CH Landverkehr 2025/30	2012
1390	FGU 2008/004_OBF	Einfluss der Grundwasserströmung auf das Quellverhalten des Gipskeupers im Belchentunnel	2012
1389	FGU 2003/002	Long Term Behaviour of the Swiss National Road Tunnels	2012
1388	SVI 2007/022	Möglichkeiten und Grenzen von elektronischen Busspuren	2012
1387	VSS 2010/205_OBF	Ablage der Prozessdaten bei Tunnel-Prozessleitsystemen	2012
1386	VSS 2006/204	Schallreflexionen an Kunstbauten im Strassenbereich	2012
1385	VSS 2004/703	Bases pour la révision des normes sur la mesure et l'évaluation de la planéité des chaussées	2012
1384	VSS 1999/249	Konzeptuelle Schnittstellen zwischen der Basisdatenbank und EMF-, EMK- und EMT-DB	2012
1383	FGU 2008/005	Einfluss der Grundwasserströmung auf das Quellverhalten des Gipskeupers im Chienbergtunnel	2012
1382	VSS 2001/504	Optimierung der statischen Eindringtiefe zur Beurteilung von harten Gussasphaltsorten	2012
1381	SVI 2004/055	Nutzen von Reisezeiteinsparungen im Personenverkehr	2012
1380	ASTRA 2007/009	Wirkungsweise und Potential von kombinierter Mobilität	2012
1379	VSS 2010/206_OBF	Harmonisierung der Abläufe und Benutzeroberflächen bei Tunnel-Prozessleitsystemen	2012
1378	SVI 2004/053	Mehr Sicherheit dank Kernfahrbahnen?	2012
1377	VSS 2009/302	Verkehrssicherheitsbeurteilung bestehender Verkehrsanlagen (Road Safety Inspection)	2012
1376	ASTRA 2011/008_004	Erfahrungen im Schweizer Betonbrückenbau	2012
1375	VSS 2008/304	Dynamische Signalisierungen auf Hauptverkehrsstrassen	2012
1374	FGU 2004/003	Entwicklung eines zerstörungsfreien Prüfverfahrens für Schweissnähte von KDB	2012
1373	VSS 2008/204	Vereinheitlichung der Tunnelbeleuchtung	2012
1372	SVI 2011/001	Verkehrssicherheitsgewinne aus Erkenntnissen aus Datapooling und strukturierten Datenanalysen	2012
1371	ASTRA 2008/017	Potenzial von Fahrgemeinschaften	2011
1370	VSS 2008/404	Dauerhaftigkeit von Betonfahrbahnen aus Betongranulat	2011
1369	VSS 2003/204	Rétention et traitement des eaux de chaussée	2012
1368	FGU 2008/002	Soll sich der Mensch dem Tunnel anpassen oder der Tunnel dem Menschen?	2011
1367	VSS 2005/801	Grundlagen betreffend Projektierung, Bau und Nachhaltigkeit von Anschlussgleisen	2011
1366	VSS 2005/702	Überprüfung des Bewertungshintergrundes zur Beurteilung der Strassengriffigkeit	2010

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1365	SVI 2004/014	Neue Erkenntnisse zum Mobilitätsverhalten dank Data Mining?	2011
1364	SVI 2009/004	Regulierung des Güterverkehrs Auswirkungen auf die Transportwirtschaft Forschungspaket UVEK/ASTRA Strategien zum wesensgerechten Einsatz der Verkehrsmittel im Güterverkehr der Schweiz TP D	2012
1363	VSS 2007/905	Verkehrsprognosen mit Online -Daten	2011
1362	SVI 2004/012	Aktivitätenorientierte Analyse des Neuverkehrs	2012
1361	SVI 2004/043	Innovative Ansätze der Parkraumbewirtschaftung	2012
1360	VSS 2010/203	Akustische Führung im Strassentunnel	2012
1359	SVI 2004/003	Wissens- und Technologientransfer im Verkehrsbereich	2012
1358	SVI 2004/079	Verkehrsanbindung von Freizeitanlagen	2012
1357	SVI 2007/007	Unaufmerksamkeit und Ablenkung: Was macht der Mensch am Steuer?	2012
1356	SVI 2007/014	Kooperation an Bahnhöfen und Haltestellen	2011
1355	FGU 2007/002	Prüfung des Sulfatwiderstandes von Beton nach SIA 262/1, Anhang D: Anwendbarkeit und Relevanz für die Praxis	2011
1354	VSS 2003/203	Anordnung, Gestaltung und Ausführung von Treppen, Rampen und Treppenwegen	2011
1353	VSS 2000/368	Grundlagen für den Fussverkehr	2011
1352	VSS 2008/302	Fussgängerstreifen (Grundlagen)	2011
1351	ASTRA 2009/001	Development of a best practice methodology for risk assessment in road tunnels	2011
1350	VSS 2007/904	IT-Security im Bereich Verkehrstelematik	2011
1349	VSS 2003/205	In-Situ-Abflussversuche zur Untersuchung der Entwässerung von Autobahnen	2011
1348	VSS 2008/801	Sicherheit bei Parallelführung und Zusammentreffen von Strassen mit der Schiene	2011
1347	VSS 2000/455	Leistungsfähigkeit von Parkierungsanlagen	2010
1346	ASTRA 2007/004	Quantifizierung von Leckagen in Abluftkanälen bei Strassentunneln mit konzentrierter Rauchabsaugung	2010
1345	SVI 2004/039	Einsatzbereiche verschiedener Verkehrsmittel in Agglomerationen	2011
1344	VSS 2009/709	Initialprojekt für das Forschungspaket "Nutzensteigerung für die Anwender des SIS"	2011
1343	VSS 2009/903	Basistechnologien für die intermodale Nutzungserfassung im Personenverkehr	2011
1342	FGU 2005/003	Untersuchungen zur Frostkörperbildung und Frosthebung beim Gefrierverfahren	2010
1341	FGU 2007/005	Design aids for the planning of TBM drives in squeezing ground	2011
1340	SVI 2004/051	Aggressionen im Verkehr	2011
1339	SVI 2005/001	Widerstandsfunktionen für Innerorts-Strassenabschnitte ausserhalb des Einflussbereiches von Knoten	2010
1338	VSS 2006/902	Wirkungsmodelle für fahrzeugseitige Einrichtungen zur Steigerung der Verkehrssicherheit	2009
1337	ASTRA 2006/015	Development of urban network travel time estimation methodology	2011
1336	ASTRA 2007/006	SPIN-ALP: Scanning the Potential of Intermodal Transport on Alpine Corridors	2010

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1335	VSS 2007/502	Stripping bei lärmmindernden Deckschichten unter Überrollbeanspruchung im Labormassstab	2011
1334	ASTRA 2009/009	Was treibt uns an? Antriebe und Treibstoffe für die Mobilität von Morgen	2011
1333	SVI 2007/001	Standards für die Mobilitätsversorgung im peripheren Raum	2011
1332	VSS 2006/905	Standardisierte Verkehrsdaten für das verkehrsträgerübergreifende Verkehrsmanagement	2011
1331	VSS 2005/501	Rückrechnung im Strassenbau	2011
1330	FGU 2008/006	Energiegewinnung aus städtischen Tunneln: Systemeevaluation	2010
1329	SVI 2004/073	Alternativen zu Fussgängerstreifen in Tempo-30-Zonen	2010
1328	VSS 2005/302	Grundlagen zur Quantifizierung der Auswirkungen von Sicherheitsdefiziten	2011
1327	VSS 2006/601	Vorhersage von Frost und Nebel für Strassen	2010
1326	VSS 2006/207	Erfolgskontrolle Fahrzeugrückhaltesysteme	2011
1325	SVI 2000/557	Indices caractéristiques d'une cité-vélo. Méthode d'évaluation des politiques cyclables en 8 indices pour les petites et moyennes communes.	2010
1324	VSS 2004/702	Eigenheiten und Konsequenzen für die Erhaltung der Strassenverkehrsanlagen im überbauten Gebiet	2009
1323	VSS 2008/205	Ereignisdetektion im Strassentunnel	2011
1322	SVI 2005/007	Zeitwerte im Personenverkehr: Wahrnehmungs- und Distanzabhängigkeit	2008
1321	VSS 2008/501	Validation de l'oedomètre CRS sur des échantillons intacts	2010
1320	VSS 2007/303	Funktionale Anforderungen an Verkehrserfassungssysteme im Zusammenhang mit Lichtsignalanlagen	2010
1319	VSS 2000/467	Auswirkungen von Verkehrsberuhigungsmassnahmen auf die Lärmimmissionen	2010
1318	FGU 2006/001	Langzeitquellversuche an anhydritführenden Gesteinen	2010
1317	VSS 2000/469	Geometrisches Normalprofil für alle Fahrzeugtypen	2010
1316	VSS 2001/701	Objektorientierte Modellierung von Strasseninformationen	2010
1315	VSS 2006/904	Abstimmung zwischen individueller Verkehrsinformation und Verkehrsmanagement	2010
1314	VSS 2005/203	Datenbank für Verkehrsaufkommensraten	2008
1313	VSS 2001/201	Kosten-/Nutzenbetrachtung von Strassenentwässerungssystemen, Ökobilanzierung	2010
1312	SVI 2004/006	Der Verkehr aus Sicht der Kinder: Schulwege von Primarschulkindern in der Schweiz	2010
1311	VSS 2000/543	VIABILITE DES PROJETS ET DES INSTALLATIONS ANNEXES	2010
1310	ASTRA 2007/002	Beeinflussung der Luftströmung in Strassentunneln im Brandfall	2010
1309	VSS 2008/303	Verkehrsregelungssysteme - Modernisierung von Lichtsignalanlagen	2010
1308	VSS 2008/201	Hindernisfreier Verkehrsraum - Anforderungen aus Sicht von Menschen mit Behinderung	2010
1307	ASTRA 2006/002	Entwicklung optimaler Mischgüter und Auswahl geeigneter Bindemittel; D-A-CH - Initialprojekt	2008
1306	ASTRA 2008/002	Strassenglätte-Prognosesystem (SGPS)	2010
1305	VSS 2000/457	Verkehrserzeugung durch Parkierungsanlagen	2009

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1304	VSS 2004/716	Massnahmenplanung im Erhaltungsmanagement von Fahrbahnen	2008
1303	ASTRA 2009/010	Geschwindigkeiten in Steigungen und Gefällen; Überprüfung	2010
1302	VSS 1999/131	Zusammenhang zwischen Bindemitteleigenschaften und Schadensbildern des Belages?	2010
1301	SVI 2007/006	Optimierung der Strassenverkehrsunfallstatistik durch Berücksichtigung von Daten aus dem Gesundheitswesen	2009
1300	VSS 2003/903	SATELROU Perspectives et applications des méthodes de navigation pour la télématique des transports routiers et pour le système d'information de la route	2010
1299	VSS 2008/502	Projet initial - Enrobés bitumineux à faibles impacts énergétiques et écologiques	2009
1298	ASTRA 2007/012	Griffigkeit auf winterlichen Fahrbahnen	2010
1297	VSS 2007/702	Einsatz von Asphaltbewehrungen (Asphalteinlagen) im Erhaltungsmanagement	2009
1296	ASTRA 2007/008	Swiss contribution to the Heavy-Duty Particle Measurement Programme (HD-PMP)	2010
1295	VSS 2005/305	Entwurfsgrundlagen für Lichtsignalanlagen und Leitfaden	2010
1294	VSS 2007/405	Wiederhol- und Vergleichspräzision der Druckfestigkeit von Gesteinskörnungen am Haufwerk	2010
1293	VSS 2005/402	Détermination de la présence et de l'efficacité de dope dans les bétons bitumineux	2010
1292	ASTRA 2006/004	Entwicklung eines Pflanzenöl-Blockheizkraftwerkes mit eigener Ölmühle	2010
1291	ASTRA 2009/005	Fahrmuster auf überlasteten Autobahnen Simultanes Berechnungsmodell für das Fahrverhalten auf Autobahnen als Grundlage für die Berechnung von Schadstoffemissionen und Fahrzeitgewinnen	2010
1290	VSS 1999/209	Conception et aménagement de passages inférieurs et supérieurs pour piétons et deux-roues légers	2008
1289	VSS 2005/505	Affinität von Gesteinskörnungen und Bitumen, nationale Umsetzung der EN	2010
1288	ASTRA 2006/020	Footprint II - Long Term Pavement Performance and Environmental Monitoring on A1	2010
1287	VSS 2008/301	Verkehrsqualität und Leistungsfähigkeit von komplexen ungesteuerten Knoten: Analytisches Schätzverfahren	2009
1286	VSS 2000/338	Verkehrsqualität und Leistungsfähigkeit auf Strassen ohne Richtungstrennung	2010
1285	VSS 2002/202	In-situ Messung der akustischen Leistungsfähigkeit von Schallschirmen	2009
1284	VSS 2004/203	Evacuation des eaux de chaussée par les bas-cotés	2010
1283	VSS 2000/339	Grundlagen für eine differenzierte Bemessung von Verkehrsanlagen	2008
1282	VSS 2004/715	Massnahmenplanung im Erhaltungsmanagement von Fahrbahnen: Zusatzkosten infolge Vor- und Aufschub von Erhaltungsmassnahmen	2010
1281	SVI 2004/002	Systematische Wirkungsanalysen von kleinen und mittleren Verkehrsvorhaben	2009
1280	ASTRA 2004/016	Auswirkungen von fahrzeuginternen Informationssystemen auf das Fahrverhalten und die Verkehrssicherheit Verkehrspsychologischer Teilbericht	2010

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1279	VSS 2005/301	Leistungsfähigkeit zweistreifiger Kreisel	2009
1278	ASTRA 2004/016	Auswirkungen von fahrzeuginternen Informationssystemen auf das Fahrverhalten und die Verkehrssicherheit - Verkehrstechnischer Teilbericht	2009
1277	SVI 2007/005	Multimodale Verkehrsqualitätsstufen für den Strassenverkehr - Vorstudie	2010
1276	VSS 2006/201	Überprüfung der schweizerischen Ganglinien	2008
1275	ASTRA 2006/016	Dynamic Urban Origin - Destination Matrix - Estimation Methodology	2009
1274	SVI 2004/088	Einsatz von Simulationswerkzeugen in der Güterverkehrs- und Transportplanung	2009
1273	ASTRA 2008/006	UNTERHALT 2000 - Massnahme M17, FORSCHUNG: Dauerhafte Materialien und Verfahren SYNTHESE - BERICHT zum Gesamtprojekt "Dauerhafte Beläge" mit den Einzelnen Forschungsprojekten: - ASTRA 200/419: Verhaltensbilanz der Beläge auf Nationalstrassen - ASTRA 2000/420: Dauerhafte Komponenten auf der Basis erfolgreicher Strecken - ASTRA 2000/421: Durabilité des enrobés - ASTRA 2000/422: Dauerhafte Beläge, Rundlaufversuch - ASTRA 2000/423: Griffigkeit der Beläge auf Autobahnen, Vergleich zwischen den Messergebnissen von SRM und SCRIM - ASTRA 2008/005: Vergleichsstrecken mit unterschiedlichen oberen Tragschichten auf einer Nationalstrasse	2008
1272	VSS 2007/304	Verkehrsregelungssysteme - behinderte und ältere Menschen an Lichtsignalanlagen	2010
1271	VSS 2004/201	Unterhalt von Lärmschirmen	2009
1270	VSS 2005/502	Interaktion Strasse Hangstabilität: Monitoring und Rückwärtsrechnung	2009
1269	VSS 2005/201	Evaluation von Fahrzeugrückhaltesystemen im Mittelstreifen von Autobahnen	2009
1268	ASTRA 2005/007	PM10-Emissionsfaktoren von Abriebspartikeln des Strassenverkehrs (APART)	2009
1267	VSS 2007/902	MDAinSVT Einsatz modellbasierter Datentransfernormen (INTERLIS) in der Strassenverkehrstelematik	2009
1266	VSS 2000/343	Unfall- und Unfallkostenraten im Strassenverkehr	2009
1265	VSS 2005/701	Zusammenhang zwischen dielektrischen Eigenschaften und Zustandsmerkmalen von bitumenhaltigen Fahrbahnbelägen (Pilotuntersuchung)	2009
1264	SVI 2004/004	Verkehrspolitische Entscheidfindung in der Verkehrsplanung	2009
1263	VSS 2001/503	Phénomène du dégel des sols gélifs dans les infrastructures des voies de communication et les pergélisols alpins	2006
1262	VSS 2003/503	Lärmverhalten von Deckschichten im Vergleich zu Gussasphalt mit strukturierter Oberfläche	2009
1261	ASTRA 2004/018	Pilotstudie zur Evaluation einer mobilen Grossversuchsanlage für beschleunigte Verkehrslastsimulation auf Strassenbelägen	2009
1260	FGU 2005/001	Testeinsatz der Methodik "Indirekte Vorauserkundung von wasserführenden Zonen mittels Temperaturdaten anhand der Messdaten des Lötschberg-Basistunnels	2009

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1259	VSS 2004/710	Massnahmenplanung im Erhaltungsmanagement von Fahrbahnen - Synthesebericht	2008
1258	VSS 2005/802	Kaphaltestellen Anforderungen und Auswirkungen	2009
1257	SVI 2004/057	Wie Strassenraumbilder den Verkehr beeinflussen Der Durchfahrtswiderstand als Arbeitsinstrument bei der städtebaulichen Gestaltung von Strassenräumen	2009
1256	VSS 2006/903	Qualitätsanforderungen an die digitale Videobild-Bearbeitung zur Verkehrsüberwachung	2009
1255	VSS 2006/901	Neue Methoden zur Erkennung und Durchsetzung der zulässigen Höchstgeschwindigkeit	2009
1254	VSS 2006/502	Drains verticaux préfabriqués thermiques pour la consolidation in-situ des sols	2009
1253	VSS 2001/203	Rétention des polluants des eaux de chausées selon le système "infilitrations sur les talus". Vérification in situ et optimisation	2009
1252	SVI 2003/001	Nettoverkehr von verkehrsintensiven Einrichtungen (VE)	2009
1251	ASTRA 2002/405	Incidence des granulats arrondis ou partiellement arrondis sur les propriétés d'ahérence des bétons bitumineux	2008
1250	VSS 2005/202	Strassenabwasser Filterschacht	2007
1249	FGU 2003/004	Einflussfaktoren auf den Brandwiderstand von Betonkonstruktionen	2009
1248	VSS 2000/433	Dynamische Eindringtiefe zur Beurteilung von Gussasphalt	2008
1247	VSS 2000/348	Anforderungen an die strassenseitige Ausrüstung bei der Umwidmung von Standstreifen	2009
1246	VSS 2004/713	Massnahmenplanung im Erhaltungsmanagement von Fahrbahnen: Bedeutung Oberflächenzustand und Tragfähigkeit sowie gegenseitige Beziehung für Gebrauchs- und Substanzwert	2009
1245	VSS 2004/701	Verfahren zur Bestimmung des Erhaltungsbedarfs in kommunalen Strassennetzen	2009
1244	VSS 2004/714	Massnahmenplanung im Erhaltungsmanagement von Fahrbahnen - Gesamtnutzen und Nutzen-Kosten-Verhältnis von standardisierten Erhaltungsmassnahmen	2008
1243	VSS 2000/463	Kosten des betrieblichen Unterhalts von Strassenanlagen	2008
1242	VSS 2005/451	Recycling von Ausbauasphalt in Heissmischgut	2007
1241	ASTRA 2001/052	Erhöhung der Aussagekraft des LCPC Spurbildungstests	2009
1240	ASTRA 2002/010	L'acceptabilité du péage de congestion : Résultats et analyse de l'enquête en Suisse	2009
1239	VSS 2000/450	Bemessungsgrundlagen für das Bewehren mit Geokunststoffen	2009
1238	VSS 2005/303	Verkehrssicherheit an Tagesbaustellen und bei Anschlüssen im Baustellenbereich von Hochleistungsstrassen	2008
1237	VSS 2007/903	Grundlagen für eCall in der Schweiz	2009
1236	ASTRA 2008/008_07	Analytische Gegenüberstellung der Strategie- und Tätigkeitsschwerpunkte ASTRA-AIPCR	2008
1235	VSS 2004/711	Forschungspaket Massnahmenplanung im EM von Fahrbahnen - Standardisierte Erhaltungsmassnahmen	2008
1234	VSS 2006/504	Expérimentation in situ du nouveau drainomètre européen	2008
1233	ASTRA 2000/420	Unterhalt 2000 Forschungsprojekt FP2 Dauerhafte Komponenten bitumenhaltiger Belagsschichten	2009
660	AGB 2008/002	Indirekt gelagerte Betonbrücken - Sachstandsbericht	2014

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659	AGB 2009/014	Suizidprävention bei Brücken: Follow-Up	2014
658	AGB 2006/015_OBF	Querkraftwiderstand vorgespannter Brücken mit ungenügender Querkraftbewehrung	2014
657	AGB 2003/012	Brücken in Holz: Möglichkeiten und Grenzen	2013
656	AGB 2009/015	Experimental verification oif integral bridge abutments	2013
655	AGB 2007/004	Fatigue Life Assessment of Roadway Bridges Based on Actual Traffic Loads	2013
654	AGB 2005-008	Thermophysical and Thermomechanical Behavior of Cold-Curing Structural Adhesives in Bridge Construction	2013
653	AGB 2007/002	Poinçonnement des pontsdalles précontraints	2013
652	AGB 2009/006	Detektion von Betonstahlbrüchen mit der magnetischen Streufeldmethode	2013
651	AGB 2006/006_OBF	Instandsetzung und Monitoring von AAR-geschädigten Stützmauern und Brücken	2013
650	AGB 2005/010	Korrosionsbeständigkeit von nichtrostenden Betonstählen	2012
649	AGB 2008/012	Anforderungen an den Karbonatisierungswiderstand von Betonen	2012
648	AGB 2005/023 + AGB 2006/003	Validierung der AAR-Prüfungen für Neubau und Instandsetzung	2011
647	AGB 2004/010	Quality Control and Monitoring of electrically isolated post-tensioning tendons in bridges	2011
646	AGB 2005/018	Interactin sol-structure : ponts à culées intégrales	2010
645	AGB 2005/021	Grundlagen für die Verwendung von Recyclingbeton aus Betongranulat	2010
644	AGB 2005/004	Hochleistungsfähiger Faserfeinkornbeton zur Effizienzsteigerung bei der Erhaltung von Kunstbauten aus Stahlbeton	2010
643	AGB 2005/014	Akustische Überwachung einer stark geschädigten Spannbetonbrücke und Zustandserfassung beim Abbruch	2010
642	AGB 2002/006	Verbund von Spanngliedern	2009
641	AGB 2007/007	Empfehlungen zur Qualitätskontrolle von Beton mit Luftpermeabilitätsmessungen	2009
640	AGB 2003/011	Nouvelle méthode de vérification des ponts mixtes à âme pleine	2010
639	AGB 2008/003	RiskNow-Falling Rocks Excel-basiertes Werkzeug zur Risikoermittlung bei Steinschlagschutzgalerien	2010
638	AGB2003/003	Ursachen der Rissbildung in Stahlbetonbauwerken aus Hochleistungsbeton und neue Wege zu deren Vermeidung	2008
637	AGB 2005/009	Détermination de la présence de chlorures à l'aide du Géoradar	2009
636	AGB 2002/028	Dimensionnement et vérification des dalles de roulement de ponts routiers	2009
635	AGB 2004/002	Applicabilité de l'enrobé drainant sur les ouvrages d'art du réseau des routes nationales	2008
634	AGB 2002/007	Untersuchungen zur Potenzialfeldmessung an Stahlbetonbauten	2008
633	AGB 2002/014	Oberflächenschutzsysteme für Betontragwerke	2008
632	AGB 2008/201	Sicherheit des Verkehrssystem Strasse und dessen Kunstbauten Testregion - Methoden zur Risikobeurteilung Schlussbericht	2010

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631	AGB 2000/555	Applications structurales du Béton Fibré à Ultra-hautes Performances aux ponts	2008
630	AGB 2002/016	Korrosionsinhibitoren für die Instandsetzung chloridverseuchter Stahlbetonbauten	2010
629	AGB 2003/001 + AGB 2005/019	Integrale Brücken - Sachstandsbericht	2008
628	AGB 2005/026	Massnahmen gegen chlorid-induzierte Korrosion und zur Erhöhung der Dauerhaftigkeit	2008
627	AGB 2002/002	Eigenschaften von normalbreiten und überbreiten Fahrbahnübergängen aus Polymerbitumen nach starker Verkehrsbelastung	2008
626	AGB 2005/110	Sicherheit des Verkehrssystems Strasse und dessen Kunstbauten: Baustellensicherheit bei Kunstbauten	2009
625	AGB 2005/109	Sicherheit des Verkehrssystems Strasse und dessen Kunstbauten: Effektivität und Effizienz von Massnahmen bei Kunstbauten	2009
624	AGB 2005/108	Sicherheit des Verkehrssystems / Strasse und dessen Kunstbauten / Risikobeurteilung für Kunstbauten	2010
623	AGB 2005/107	Sicherheit des Verkehrssystems Strasse und dessen Kunstbauten: Tragsicherheit der bestehenden Kunstbauten	2009
622	AGB 2005/106	Rechtliche Aspekte eines risiko- und effizienzbasierten Sicherheitskonzepts	2009
621	AGB 2005/105	Sicherheit des Verkehrssystems Strasse und dessen Kunstbauten Szenarien der Gefahrenentwicklung	2009
620	AGB 2005/104	Sicherheit des Verkehrssystems Strasse und dessen Kunstbauten: Effektivität und Effizienz von Massnahmen	2009
619	AGB 2005/103	Sicherheit des Verkehrssystems / Strasse und dessen Kunstbauten / Ermittlung des Netzrisikos	2010
618	AGB 2005/102	Sicherheit des Verkehrssystems Strasse und dessen Kunstbauten: Methodik zur vergleichenden Risikobeurteilung	2009
617	AGB 2005/100	Sicherheit des Verkehrssystems Strasse und dessen Kunstbauten Synthesebericht	2010
616	AGB 2002/020	Beurteilung von Risiken und Kriterien zur Festlegung akzeptierter Risiken in Folge aussergewöhnlicher Einwirkungen bei Kunstbauten	2009