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Long Term Behaviour of the Swiss National Road Tunnels

Comportement à long terme des tunnels des routes nationales

Langzeitverhalten der Schweizerischen Nationalstrassen-Tunnels

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

Dr F. Sandrone
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Mandat de recherche FGU 2003/002 sur demande de Fachgruppe Untertagbau (FGU)

December 2012 1389

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Résumé

Ce projet, soutenu par l'Office Fédéral des Routes (OFROU), traite du comportement à long terme des tunnels autoroutiers suisses et de leurs principales pathologies.

Etant donné qu'un tunnel est un ensemble constitué d'un revêtement et d'un massif rocheux, il a été nécessaire de considérer les effets de la dégradation de ces deux éléments. Les processus de dégradation ont été divisés en trois classes principales selon le type et les effets du processus: vieillissement, détérioration et autres processus mécaniques et physiques.

Les conditions initiales (c.-à-d. la profondeur du tunnel, les détails de construction tels que le système d'étanchéité, et l'âge de la structure, ainsi que les conditions géologiques et hydrogéologiques), les conditions d'exploitation (par ex. pollution par le trafic, sels de déverglaçage) et les résultats d'inspections (désordres) ont été pris en considération pour déterminer les effets à long terme des processus de dégradation.

Après une revue détaillée de la littérature, et sur la base de conseils fournis par des responsables de la maintenance de tunnels, une base de données (Tunnel Data Base), ainsi qu'un formulaire technique pour le recueil de ces données, ont été élaborés. Plusieurs sources d'informations ont été compilées. Hormis des informations générales, la base de données recueille des informations techniques, des données sur la construction et des résultats d'inspections des tunnels, émanant directement de la consultation des archives cantonales et des responsables de la maintenance des routes nationales suisses. En outre des informations géologiques et hydrogéologiques ainsi que des données de trafic ont été intégrées à l'aide d'outils S.I.G.

Une analyse fouillée des données collectées a été effectuée (168 tunnels dont 122 documentés avec les désordres relevés lors des inspections principales). Des désordres typiques affectant les tunnels des routes nationales suisses ont été identifiés. De plus, à l'aide d'une analyse des correspondances (méthode de statistiques multivariées), il a été possible de déterminer parmi les diverses caractéristiques des tunnels quels sont les principaux facteurs impliqués dans le développement des désordres. Tant le potentiel de dégradation, dû aux conditions initiales du tunnel, que le taux de dégradation, dû aux conditions d'exploitation, ont été étudiés. Cette analyse démontre que les désordres identifiés pendant les inspections et qui affectent le revêtement peuvent être causés par la dégradation du massif rocheux et par son comportement différé, mais aussi par les conditions environnementales et d'exploitation (par ex. trafic, sels de déverglaçage). Par exemple, il a été possible de montrer comment :

- Des volumes importants de trafic, liés à l'utilisation des sels de déverglaçage peuvent être particulièrement agressifs pour le béton du revêtement. L'altération du revêtement et la corrosion due à l'action des sels de déverglaçage affectent principalement les parties inférieures des parois du tunnel.
- La composition chimique de l'eau du massif, en l'absence de système d'étanchéité, la faible qualité des matériaux de construction et l'atmosphère agressive à l'intérieur du tunnel accélèrent le processus d'altération. Les symptômes principaux de cette pathologie sont les efflorescences et la corrosion du béton (par ex. attaque des sulfates, lessivage du calcium) qui affectent en particulier la voûte du tunnel.
- Les effets dus au comportement poussant, lié à de mauvaises conditions géologiques et qui se développe à certaine profondeur, peuvent être réduits par un design approprié du tunnel (par ex., soutènement et géométrie du tunnel).

Connaissant les conditions initiales et les taux de dégradation du revêtement et du massif rocheux, il est possible d'évaluer l'évolution dans le temps des conditions de stabilité du tunnel. Plusieurs exemples d'application ont été proposés dans le cadre de la méthode convergence-confinement. Des modèles mathématiques ont été utilisés pour décrire divers processus différés de dégradation (par ex. des lois de comportement visco-élastique et visco-plastique, ou une diminution de l'épaisseur du revêtement ou de sa résistance). Les résultats, dont l'évolution des conditions d'équilibre, ont été analysés en termes de facteur de sécurité pour le tunnel.

En raison des hypothèses restrictives sur lesquelles est basée la méthode convergence-confinement, le but n'est pas de remplacer une analyse de stabilité détaillée, qui reste nécessaire pour déterminer les conditions de stabilité à long terme d'un tunnel, mais d'identifier les principaux facteurs impliqués dans le changement des conditions d'équilibre dû aux processus de dégradation.

Les résultats montrent clairement l'importance d'une caractérisation détaillée du tunnel, à la fois lors de sa construction et durant son exploitation (au moyen des résultats d'inspection), afin d'obtenir un diagnostic complet du processus de dégradation et des principaux facteurs en jeu. En fait, il est nécessaire de tenir en compte des pathologies différées qui affectent à la fois le massif et le revêtement de béton pour prédire correctement le comportement à long terme du tunnel.

Cette étude conduit à certaines recommandations pour aider à établir les conditions à long terme du tunnel et améliorer les pratiques de conservation :

1. Les résultats d'inspection et l'observation des désordres, ainsi qu'une caractérisation détaillée à l'échelle locale de l'ouvrage sont nécessaires pour interpréter correctement les phénomènes de dégradation du tunnel. Actuellement le problème principal rencontré lors

de la récolte des données a été le manque de standardisation de l'enregistrement des résultats d'inspection. Pour améliorer le système actuel il est recommandé de documenter les désordres observés par des dessins ou des enregistrements par camera qui montrent la position des désordres et leur gravité. De cette façon, à travers une mise à jour régulière de l'information, il est possible de suivre l'évolution de chaque désordre.

2. Vu que les résultats de l'analyse des données montrent la forte influence des conditions géologiques et hydrogéologiques, ainsi que des détails de constructions sur la dégradation du tunnel, une documentation détaillée à l'échelle locale de l'ouvrage doit être fournie avec les rapports d'inspection. Pour simplifier et améliorer l'interprétation des désordres observés, il est recommandé de représenter ces informations géologiques, hydrogéologiques et les détails de construction sous la forme d'un profil de tunnel utilisable avec les résultats d'inspection.
3. Vu que la position du tunnel détermine non seulement son environnement géologique et hydrogéologique mais aussi les conditions climatiques et d'exploitation (c.à.d. le trafic et l'utilisation de sels de déverglaçage, en particulier), la correspondance entre les résultats des analyses S.I.G. et les désordres observés permet de considérer les S.I.G. comme un outil plein de promesses pour la détection préliminaire des pathologies potentielles des tunnels. De plus, une caractéristique intéressante de ces outils est la possibilité de gérer toute l'information stockée dans le Tunnel Data Base pour une analyse immédiate et une identification des problèmes majeurs. Au fait, une analyse préliminaire et " brute " peut se révéler particulièrement intéressante pour donner la priorité aux interventions et décider de la maintenance préventive du tunnel.
4. La procédure d'analyse de la base de données TDB utilisée pour ce travail est inspirée des travaux déjà entrepris à l'OFROU pour évaluer les conditions des ponts des Routes Nationales (KUBA-MS). Actuellement le Tunnel Data Base ne contient pas assez d'informations détaillées sur des inspections successives. Donc, avant de devenir un outil performant pour la maintenance des tunnels, des résultats d'inspections successives sont nécessaires pour estimer l'évolution dans le temps des désordres observés. L'évolution des désordres peut aussi être analysée au moyen des mesures d'auscultation. Par exemple, quand un tunnel est excavé dans un massif au comportement différé, le suivi des contraintes et des déformations dans le temps peut se révéler très utile pour prédire le comportement à long terme du revêtement. Cependant pour les tunnels routiers suisses actuels, bien que fortement recommandé surtout en cas de désordres évidents, l'auscultation du profil ne fait pas partie des techniques de contrôle communes lors des inspections principales.
5. Pour une planification complète de la maintenance, une analyse des coûts sur la durée de l'ouvrage (life-cycle cost analysis) peut être faite en comparant les opérations de remise en état à court terme et à long terme. Cette analyse doit intégrer toutes les

informations sur les conditions du tunnel, basées principalement sur les résultats d'inspections, et une évaluation socio-économique, qui dépend surtout de l'importance de l'ouvrage sur l'ensemble du réseau routier. En suivant la procédure d'analyse des données utilisée pour identifier les facteurs qui influencent surtout la dégradation du tunnel, une note globale, bien que simple, ne peut être considérée comme représentative pour décrire les conditions du tunnel. Bien que cet aspect ne soit pas traité dans ce travail, quelques considérations peuvent être faites en prévision de la création d'un index pour noter les conditions actuelles des tunnels. Par exemple, une approche intéressante et différente est l'utilisation de plusieurs indexes qui décrivent les conditions de l'ouvrage par rapport aux désordres identifiés. La localisation et l'évaluation de la gravité de la pathologie (c. à d. l'importance de la zone endommagée) peuvent être utilisées pour pondérer chaque index. De plus, l'identification des scenarios principaux qui déterminent la vitesse de dégradation peut aider à focaliser l'attention (et les moyens) sur les processus majeurs de dégradation. Finalement, après un bon diagnostic des conditions du tunnel, les facteurs sociaux et économiques détermineront la décision du maître de l'ouvrage.

MOTS-CLES:

Tunnels routiers, Pathologies, Comportement à long terme, Base de données, Analyse des Correspondances, Méthode convergence-confinement

Zusammenfassung

Dieses Projekt hat zum Zweck, mit der Unterstützung vom Bundesamt für Strassen (ASTRA) das langfristige Verhalten von Autobahntunnel in der Schweiz zu untersuchen und ihre Hauptbeschädigungserscheinungen (in dieser Arbeit als "Pathologien" bezeichnet) zu charakterisieren.

Ein Tunnel ist als eine Zusammensetzung zwischen einer Auskleidung und dem umliegenden Fels zu verstehen. In dieser Hinsicht was es entscheidend, die spezifischen Auswirkungen von gezielten Beschädigungen dieser zwei Elementen zu analysieren. Die Beschädigungsprozesse wurden somit in den drei folgenden Hauptkategorien unterteilt, unter Berücksichtigung des Veralterungsmusters und der resultierenden Auswirkungen: Veralterung, Verwitterung sowie weitere mechanische und physikalische Prozesse.

Um die langfristigen Auswirkungen der Beschädigungsprozessen zu bestimmen, wurden folgende Rahmenbedingungen berücksichtigt: die Tunneltiefe, Baumerkmale (z.B. das Abdichtungssystem, der Strukturalter, oder die geologischen und hydrogeologischen Verhältnisse), der Betrieb (z.B. Effekte der Verschmutzung durch den Verkehr oder des Nutzens von Enteisungssalz) und die Inspektionsresultate (Pathologien).

Nach einem ausführlichen Literaturstudium und gestützt auf Vorschläge von Tunnelwartungs-Fachleuten, wurden eine Datenbank (Tunnel Data Base) und ein technisches Formular zur Datenerfassung geschaffen. Es wurden mehrere Informationsquellen verarbeitet. Die Datenbank beinhaltet allgemeine und technische Informationen, Baudaten und Tunnelkontrollresultate, welche direkt aus den Kantonsarchiven, oder von Wartungsfachleuten der Schweizer Autobahnen stammen. Auch geologische und hydrogeologische Informationen, zusammen mit Verkehrsdaten, wurden mit Hilfe mittels G.I.S. Werkzeugen integriert.

Es wurde eine durchgreifende Analyse der gesammelten Daten durchgeführt (168 Tunnel, wovon 122 Tunnel mit während Hauptkontrollen aufgenommenen Pathologien). Die typischen Pathologien der Schweizer Autobahntunnel wurden identifiziert. Ausserdem hat eine Korrespondenzanalyse (multivariate Statistikmethode) es erlaubt, die für die Entwicklung von Pathologien verantwortlichen Hauptfaktoren herauszukristallisieren. Es wurden sowohl das Beschädigungspotential (bedingt durch den Ausgangszustand), wie die betriebsbedingte Beschädigungsrate untersucht. Diese Analyse bewies, dass die durch Inspektionen identifizierten Auskleidungspathologien durch das Langzeitverhalten des Gebirges, sowie durch die Umwelt- und Betriebsbedingungen (z.B. Verkehr, Enteisungssalz) verursacht werden. Es konnte z.B. gezeigt werden, wie:

- Hohe Verkehrsaufkommen, zusammen mit dem Gebrauch von Enteisungssalz, besonders aggressiv für den Verkleidungsbeton sein können. Die Verwitterung der Verkleidung und die Korrosion wegen des Enteisungssalzes betreffen hauptsächlich den unteren Teil der Tunnelwände.
- Die chemische Zusammensetzung des Grundwassers, bei Fehlender Wasserdichtung, die geringe Qualität der Baumaterialien und die aggressive Tunnelluft den Verwitterungsprozess beschleunigen. Die Hauptsymptome dieser Pathologien sind Betonausblühungen und -korrosion (z.B. Angreifen durch Sulfate, Auslaugen des Kalziums), welche hauptsächlich das Tunnelgewölbe betreffen.
- Die negativen Auswirkungen eines druckhaften Gebirges in ungünstigen geologischen Bedingungen und unter einer hohen Überlagerung durch ein passendes Tunneldesign (z.B. Tunnelausbau und -geometrie) verringert werden können.

Sind Ausgangszustand und Beschädigungsrate des Gebirges und der Verkleidung bekannt, ist es möglich, die zeitliche Entwicklung der Tunnelstabilität abzuschätzen. Mehrere Anwendungsbeispiele wurden im Rahmen der Konvergenzmethode vorgeschlagen. Mathematische Modelle wurden zur Beschreibung verschiedener zeitlich verzögerter Verwitterungsprozesse angewandt (z.B. viskoelastisches, bzw. viskoplastisches Verhalten, oder Minderung der Verkleidungsstärke oder -festigkeit). Die Resultate, inkl. die zeitliche Entwicklung der Gleichgewichtzustände, wurden mittels Tunnel-Sicherheitsfaktoren analysiert.

Da die Konvergenzmethode auf sehr restriktive Hypothesen basiert, kann sie eine detaillierte Stabilitätsanalyse nicht ersetzen, und diese bleibt für die Bestimmung der langfristigen Tunnelstabilität absolut notwendig. Die Konvergenzmethode ermöglicht es aber, die Hauptfaktoren der Gleichgewichtsänderungen infolge Beschädigungsprozesse zu identifizieren.

Die Ergebnisse haben eindeutig gezeigt, dass eine detaillierte Beschreibung des Tunnels, sowohl beim Bau wie während des Betriebs (durch Inspektionsergebnisse) für eine komplette Diagnose der Beschädigungsprozesse und für die Identifikation der entsprechenden Hauptfaktoren erforderlich ist. Für eine korrekte Einschätzung des Tunnel-Langzeitverhaltens ist es notwendig, die zeitlich verzögerten Pathologien, welche sowohl das Gebirge wie die Betonverkleidung betreffen, zu berücksichtigen.

Diese Studie führte zu einigen Empfehlungen, die dazu helfen werden, die langfristigen Tunnelzustände einzuschätzen und die Instandhaltung zu verbessern:

1. Die Inspektionsresultate und Pathologiebeobachtungen, zusammen mit einer ausführlichen Tunnelbeschreibung in lokalem Massstab, sind für eine korrekte Deutung der Tunnelbeschädigung notwendig. Bis jetzt war das Hauptproblem der Datensammlung

die mangelhafte Standardisierung bei der Eintragung der Inspektionsresultate. Um das aktuelle System zu verbessern wird empfohlen, die beobachteten Pathologien mittels Skizzen oder Kameraaufnahmen, welche sowohl Position und Ausmass zeigen, zu dokumentieren. Auf diese Weise wird es möglich sein, durch eine regelmässige Aktualisierung der Informationen, der Entwicklung jeder Pathologie zu folgen.

2. Da die Resultate der Datenanalyse den bedeutenden Einfluss der geologischen und hydrogeologischen Bedingungen, sowie der Baudetails auf die Tunnelbeschädigung beweisen, sollte eine detaillierte Tunneldokumentation mit den Inspektionsberichten abgeliefert werden. Um die Interpretation der beobachteten Pathologien zu erleichtern und zu verbessern wird empfohlen, diese geologischen und hydrogeologischen Informationen, sowie die Baudetails in Form eines Tunnelprofils, das zusammen mit den Inspektionsresultaten verwendet werden kann, darzustellen.
3. Die Tunnellage bestimmt nicht nur das geologische und hydrogeologische Umfeld, sondern auch die klimatischen und betrieblichen Bedingungen (d.h. insbesondere das Verkehrsaufkommen und den Einsatz von Enteisungssalz). Das Einstimmen der Resultate von G.I.S. Analysen mit den beobachteten Pathologien erlaubt es also, die G.I.S. als ein vielversprechendes Werkzeug für das vorzeitige Erkennen potentieller Tunnelpathologien zu betrachten. Eine weitere interessante Eigenschaft dieses Werkzeugs ist die Möglichkeit, eine unmittelbare Analyse mit Identifikation der Hauptprobleme anhand der gesamten Tunneldatenbank-Informationen durchzuführen. In der Tat kann sich eine vorgängige Analyse als sehr interessant erweisen, um Prioritäten bei den Interventionen zu setzen und über vorbeugende Massnahmen zu entscheiden.
4. Das für diese Studie angewandte Untersuchungsverfahren der TDB Datenbank basiert auf dem aktuellen ASTRA-Verfahren zur Beurteilung der Nationalstrassen-Brückenzustände (KUBA-MS). Zurzeit enthält die Tunnel-Datenbank leider noch nicht genügend ausführliche Informationen über aufeinanderfolgende Tunnelkontrollen. Bevor es zu einem leistungsfähigen Tunnelwartungs-Werkzeug werden kann, sind demzufolge Resultate von sukzessiven Kontrollen nötig, damit die Zeitentwicklung der beobachteten Pathologien abgeschätzt werden kann. Diese Zeitentwicklung kann auch mittels Überwachungen und Kontrollmessungen analysiert werden. Wenn z.B. ein Tunnel in einem Gebirge mit zeitabhängigem Verhalten aufgefahren wird, kann sich die zeitliche Entwicklung der Spannungen und Verformungen als sehr nützlich erweisen, um das Langzeitverhalten der Verkleidung vorherzusagen. Bei den heutigen Schweizer Strassentunnel gehört jedoch die Profilüberwachung nicht zu den gemeinsamen Hauptinspektions-Kontrolltechniken, obwohl sie besonders bei offensichtlichen Pathologien stark empfohlen wird.
5. Für eine komplette Wartungsplanung, kann eine Kostenanalyse über den gesamten Lebenszyklus des Bauwerkes (life-cycle cost analysis) durchgeführt werden, indem

man die kurzfristigen mit den langfristigen Rehabilitationskosten vergleicht. Diese Analyse muss alle Informationen über den Tunnelzustand, der hauptsächlich auf den Kontrollresultaten basieren, und eine soziologisch-ökonomische Abschätzung, welche von der Wichtigkeit des Tunnels im gesamten Strassennetz abhängt, beinhalten. Für das verwendete Datenanalyse-Verfahren zur Identifizierung der Tunnelbeschädigungs-Hauptfaktoren genügt eine globale Bewertung, trotz seiner Einfachheit, nicht zur Beschreibung des aktuellen Tunnelzustandes. Obwohl das Thema hier nicht behandelt wurde, können einige Betrachtungen in der Perspektive eines zukünftigen Tunnelbewertungs-Indexes gemacht werden. Eine interessante Alternative wäre z.B. die Verwendung von mehreren Indexen zur Beschreibung der Tunnelzustände in Betracht der aufgenommenen Pathologien. Lage und Ausmass der Pathologien (d.h. Ausmass der beschädigten Zone) können zur Gewichtung der einzelnen Indexe dienen. Ausserdem kann die Identifizierung der die Beschädigungsgeschwindigkeit bestimmenden Hauptszenarien dazu helfen, die Aufmerksamkeit (und die Mittel) auf die Hauptbeschädigungsprozesse zu fokalisieren. Nach einer erfolgreichen Diagnose der Tunnelbedingungen werden schlussendlich die sozialen und ökonomischen Faktoren die Entscheidung des Tunnelinhabers bestimmen.

SCHLÜSSELWÖRTER:

Strassentunnel, Pathologien, langfristiges Verhalten, Datenbank, Korrespondenz-Analyse, Konvergenzmethode

Abstract

This project focuses on the long term behaviour of National Road tunnels and on their main pathologies.

As a tunnel is composed by lining structure and rock mass, it has been necessary to consider the effects of degradation on both of them. Degradation processes have been divided into three main classes depending on process type and effects: ageing, weathering, and other mechanical and physical processes.

Initial conditions (i.e. tunnel depth, construction features as waterproofing system, and age of the structure, together with geological and hydrogeological conditions), operation conditions (e.g. traffic pollution and de-icing salts), and inspection results (disorders) have been taken into account for determining the long term effects of the degradation processes.

After a detailed literature review and based on advices from tunnel inspectors, a data base (Tunnel Data Base) and a technical form for data collection have been created. Several information sources have been compiled. Apart from general information, the data base stores technical information, data about construction and tunnel inspections results, collected directly by consulting cantonal archives and people responsible for the Swiss National Roads. Also geological and hydrogeological information, together with traffic data, have been integrated using G.I.S. tools.

A comprehensive analysis of the collected data has been performed (168 tunnels; 122 tunnels with disorders data from principal inspections). Typical disorders affecting Swiss National Roads tunnels have been identified. Moreover, using Correspondence Analysis (multivariate statistics method) it has been possible to select, within all the tunnel features, the main factors involved in the development of disorders. Degradation potential (due to the tunnel initial conditions) and rate (due to operation conditions) have been investigated. This analysis shows that lining disorders, identified during tunnel inspections, may be caused by rock mass degradation and delayed behaviour, as well as, tunnel environment and operation conditions (e.g. traffic, de-icing salts). For instance it has been shown how:

- High traffic volumes, together with the use of de-icing salts can be particularly aggressive for the concrete lining. Indeed, lining weathering and corrosion due to de-icing salts attack mainly affect the lower part of the tunnel side walls.
- The chemical composition of ground water, together with the absence of waterproofing system, the poor quality of building materials and the aggressive atmosphere inside the tunnel accelerate lining weathering rate. The main symptoms of this pathology are

efflorescence and concrete corrosion (e.g. sulphates attack, calcium leaching...) that affect mainly tunnel crown.

- The effects of squeezing potential due to bad geological conditions, and developing at a certain depth, may be reduced by an appropriate tunnel design (e.g. supporting structure and tunnel geometry).

Based on the initial conditions and the degradation rate of both rock mass and lining, it is possible to assess the evolution with time of the tunnel stability conditions. Some examples illustrating the application of this methodology have been proposed in the framework of the convergence-confinement method. Within the basic assumptions of the method, mathematical models were used for describing the time dependent degradation processes (e.g. rock mass ageing described by viscoelastic or viscoplastic behaviours, lining weathering described by thickness or strength reductions). The results (i.e. changing equilibrium conditions) have been interpreted in terms of tunnel Safety Factor.

Due to the restrictive assumptions on which the convergence-confinement method is based, the aim is not to replace detailed stability analyses, which remain necessary for determining the long term tunnel stability conditions, but to identify the main factors involved in changing equilibrium conditions due to degradation processes.

The results show clearly the importance of a detailed characterisation of the tunnel, since its construction and during its service life (by means of inspection results), in order to perform a complete diagnostic of the degradation processes and of the main factors involved. Indeed, taking into account the delayed pathologies that affect both the rock mass and the concrete lining is necessary for a correct assessment of the tunnel long term behaviour.

This study led to some recommendations for helping with long term tunnel conditions assessment and improving tunnel conservation practices:

1. Inspection results and disorders observations, together with a detailed tunnel characterisation at the local scale are necessary for correctly interpreting tunnel degradation phenomena. At present, the main problem encountered during data collection has been the lack of a standardised way of recording inspection results. To improve the actual system it is recommended to document disorders observations by means of sketches or camera recording showing their position and severity. In this way, through a regular update of the information it is possible to follow the evolution of each disorder.
2. Since data analyses results show the great influence of geological and hydrogeological conditions, together with construction details, on tunnel degradation, an accurate documentation at the tunnel local scale should be available together with inspection reports. In order to facilitate and improve the interpretation of observed disorders, it is

recommended to represent those geological, hydrogeological and construction data, in the form of a tunnel profile to be used together with the inspection sketches.

3. Since tunnel location determines not only its geological and hydrogeological environment, but also climatic and operation conditions (e.g. in particular, traffic volumes and use of de-icing salts), the agreement of G.I.S. analyses results and observed disorders allows considering G.I.S. as a promising tool for preliminary detection of tunnels potential pathologies. Moreover, an interesting feature of this tool is the possibility of managing all the information stored in the Tunnel Data Base for immediate data analyses and major problems identification. Indeed, a preliminary rough data analysis may reveal quite interesting for prioritising interventions and/or deciding about preventive maintenance.
4. The TDB analysis procedure used in this work, is inspired from what is already done by OFROU for evaluating National Roads bridges conditions (KUBA-MS). At present, the Tunnel Data Base does not contain enough and detailed information about tunnels successive inspections. Thus, before becoming a tool for tunnel maintenance decisions, successive inspections results are required for estimating time evolution of observed disorders. Disorders evolution can be also analysed by means of monitoring measurements interpretation. For example, when a tunnel is excavated in evolutive rock masses, the follow up of stresses and strains with time may reveal quite useful in assessing the long term behaviour of the lining. However, for Swiss Road tunnels at present, though highly recommended in case of evident disorders, profile monitoring is not included within common tunnel survey techniques during principal inspections.
5. For a complete maintenance planning, a life-cycle cost analysis can be made by comparing repair options in both short and long term rehabilitation. The analysis should integrate tunnel conditions information, which is mainly based on principal inspection results, and a socio-economical evaluation, which mainly depends on the tunnel importance on the whole road network. Following the data analysis procedure used for identifying the main factors that influence tunnel degradation, a global mark, in spite of its simplicity, can not be considered relevant for describing actual tunnel conditions. Though this topic is not treated in this work, some considerations can be done in the perspective of indexing actual tunnel conditions. For example, an interesting/alternative approach could be the use of several indexes describing tunnel conditions with respect to the identified disorders. Pathology location and severity (i.e. affected zone extension) could be used as an internal weight for each index. Moreover, the identification of main scenarios that determine degradation rate could help in focusing attention (and means) on major degradation processes. Finally, after a good diagnostic of the tunnel conditions, social and economical factors will determine the tunnel owner decision.

KEYWORDS:

Road Tunnels, Pathologies, Long term behaviour, Data Base, Correspondence Analysis, Convergence-confinement method

1. Introduction

With bridges and viaducts, tunnels are classified as “work of art” [103] and they can be considered one of the most expensive infrastructures of the traffic network. People have been excavating tunnels for more than 4000 years now. Nowadays, in Europe and North America the traffic network is quite well developed and the majority of tunnels has been already excavated. Thus, considering the age of road and railway infrastructures, the main problem for network owners deals more with old tunnels maintenance, rehabilitation and refurbishment than with new tunnels construction.

After a brief overview on tunnel construction in the past, by comparing the tunnel incidence in several countries, this chapter focuses on the importance of tunnels for transportation networks. Then, the Swiss Roads network and the main features that characterise National Roads tunnels are detailed.

1.1 Brief historical overview on traffic tunnels

As reported by [142], the oldest tunnel constructed with communication purposes was built in 2000 b.C. in Babylon. Then, in the period of ancient Greeks and Romans and during the Middle Ages, the majority of tunnels were constructed for military and civil purposes as tombs and aqueducts. The development in traffic tunnels excavation, in particular for navigation purposes, began during the XVII century. Late in that century, the manual excavation method with hammer and wedges was replaced by rock blasting with the introduction of gunpowder. Then, since the second half of the XIX century, the increase in transportation needs required well developed railway and road networks and consequently more tunnels.

Fig. 1.1. The Urnerloch (XVIII century): first tunnel excavated in the Swiss Alps for communication purposes (Source: www.a2-gottardo.ch).

In Switzerland the very first tunnel, a Swiss Alpine pass, called Urnerloch (see Figure 1.1), was built at the beginning of XVIII century, in 1707. This tunnel was made to facilitate the passage over the St. Gotthard. It was about 61 meters long. Then, several tunnels were excavated during the railway network construction in the second half of the XIX century [Kovári et Descoeudres, 2001]. In that century a lot of improvements were done in the field of mining and tunnelling activities. During the Mont Cenis Tunnel construction, between Italy and France, in 1861, Sommellier introduced the pneumatic rock drill and, in 1867, dynamite, discovered by Nobel in 1864, was used for the first time for rock blasting. In the same period, the introduction of shields allowed the excavation in difficult geological conditions. Thanks to all those improvements in excavation techniques, tunnel construction became easier and faster and the traffic network grew. The end of the XIX century in Switzerland was characterized by the excavation of the most important alpine railway tunnels as Gotthard, in 1881, Simplon, in 1898, and Lötschberg, in 1906. In the 1950s, after the Second World War, also the road network began developing a lot. In the same period, the mechanised excavation method was improved by the introduction of the first tunnel boring machine (Robbins Machine in 1954). In the 1960s, the Swiss National Roads developed rapidly [48]. Due to the mountainous topography, the road network construction required the excavation of a great number of tunnels. The first road tunnel under the Alps was the Great St. Bernard (about 5900 m long) linking Switzerland and Italy since 1964.

1.2 Traffic tunnels nowadays

By compiling different sources, as summarised in Table 1.1, it has been possible to compare for several countries:

- the transportation networks total length (10^3 km), in particular motorways/National Roads and railways,
- the tunnel length (10^3 km) for each type of network,
- the tunnel incidence (%) on the total length of the network,
- the number of tunnels,
- the average age of structures (years),
- the age of the oldest tunnel (years).

Due to the lack of information it is not possible to evaluate the total tunnel length for road and railway networks in each country considered. Some problems have been encountered also in estimating the average age of tunnels and in finding information about the oldest tunnel in service. Thus, in some cases, the data reported in italic in Table 1.1 do not cover the whole information range, in particular:

- the considered road tunnels have a minimum tunnel length of 500 m in Portugal and Spain, 1000 m in USA, 2000 m in Switzerland and 3000 m in Japan,
- the considered railway tunnels have a minimum tunnel length of 1000 m in Portugal, 2000 m in Austria and USA and 3000 m in China.

Anyway, some preliminary considerations can be done:

1. The tunnel incidence reflects clearly the country topography. Indeed, for mountainous countries as Switzerland, Italy and Japan, for example, the tunnel incidence is quite high for both road and railway networks. This may influence maintenance costs and tasks [103].
2. By looking at road and railway tunnels age, it is evident that road tunnels are quite “younger” compared to railway ones. This difference is more clear by comparing construction material and techniques, together with all other features that depend on construction year. Consequently, both construction and operation conditions may change a lot from one kind of structure to the other one.
3. If compared to railway structures, the relative “young age” of road tunnels may delay the symptoms manifestation of long term pathologies. This results, for some countries, in a lack of knowledge about degradation processes and, consequently, about efficient conservation procedures.

Tab. 1.1. Road and railway tunnels incidence and average age of tunnels in European and Extra-European Countries. Network Type: M = Motorways; NR = National Roads. Data in italic cover only a limited range of information as detailed in the text. Data Sources are detailed in the table's footnote.

Country	**Network**	**Total Length** (10^3 **km**)	**Tunnel Length** (10^3 **km**)	**Tunnel Incidence (%)**	**No. Tunnels**	**Average Age (years)**	**Oldest Tunnel (years)**
Switzerland	M/NR	1.9	0.198	10.4	213	30-35	-
	All Roads	71.6	0.15	0.21	39	>50	-
	Railway	2.9	0.23	7.9	290	73%>80	150
France	M/NR	38.1	0.047	0.12	96	-	150
	All Roads	999.6	0.186	0.019	574	20%>100	150
	Railway	27.8	0.62	2.2	1'551	87%>100	180
Italy	M/NR	52.6	0.906	1.7	741	40	-
	Railway	16.1	1.55	9.6	-	-	-
UK	M/NR	51.8	0.032	0.062	49	40	-
	Railway	16.4	0.32	2.0	-	96%>80	-
Austria	M/NR	11.9	0.16	1.3	74	-	-
	Railway	6.0	0.064	1.1	12	-	-
Portugal	M/NR	11.9	0.16	1.3	74	-	-
	Railway	3.1	0.003	0.10	3	>100	-
Spain	M/NR	26.2	0.195	0.7	139	-	-
	Railway	11.1	0.5	4.5	-	59%>80	-
China	M/NR	29.7	0.13	0.44	66	-	-
	Railway	73.0	0.19	0.26	25	-	-
USA	M/NR	90.7	0.04	0.044	47	>50	-
	Railway	207.2	0.044	0.021	5	50-100	>100
Japan	M/NR	6.9	0.25	3.6	60	30-40	-
	Railway	23.7	4.6	19.4	-	-	50

Sources:
"The World's longest Tunnel Page" © 2003 (http://home.no.net/lotsberg/index_fr.html);
R. Käppeli, R+R Burger und Partner AG Ingenieure und Ökonomen, personal communication;
CETu, personal communication and [31] (http://www.cetu.equipement.gouv.fr);
European Road Association, 2006 (http://www.erf.be);
Office fédéral de la statistique - OFS, 2006 (http://www.bfs.admin.ch)

1.3 Swiss roads and tunnels

Due to its central position in Europe, Switzerland plays an important role in terms of European traffic and transports. Today, the road traffic network has an extension of more than 70'000 km with a circulation of traffic and merchandise which has almost doubled during the last 30 years [105].

In Switzerland, there are three classes of National Roads, (Routes Nationales, RN) which are distinguished by the letter A followed by a number:

- First class motorways and highways, specific for high-speed traffic, usually with few if any intersections, limited accesses/exits and a divider between lanes for traffic moving in opposite directions. They are characterised by having more than two lanes in each direction.

- Second class highways, smaller than motorways, characterized by a high-speed traffic, having few intersections, limited accesses/exits and a divider between lanes for traffic moving in opposite directions.
- Third class roads, avoiding crossing towns or cities, characterised by having limited accesses/exits but no hard shoulder (i.e. security lane).

The Swiss National Roads, today, have an extension of about 1900 (1893) km and they are managed by both Cantons and Confederation, at least until the year 2008.

In Figure 1.2 it is also represented the Main Roads network (Routes Principales Cantonales, RP). Those roads are distinguished by the letter H (i.e. Hauptstrasse) followed by a number. Based on their location, Main Roads are also distinguished by another letter, i.e.:

- T (i.e. Talstrasse) followed by a number for plain roads,
- J (i.e. Jurastrasse) followed by a number for Jura mountains roads,
- A (i.e. Alpenstrasse) followed by a number for Alpine roads.

Main Roads have an extension of about 2300 km and they are entirely managed/controlled by Cantons, with a grant of the Confederation (more details in Chapter 3).

Fig. 1.2. Swiss Roads sketch as proposed by [106]: National Roads are represented by thicker numbered lines, while Main Roads by thinner lines (source: Routes et Trafic - http://www.astra.admin.ch).

In Switzerland, the land mountainous topography made necessary the construction of several tunnels. Figure 1.3 shows the evolution of excavated length per decade for Swiss roads tunnel. As documented by [101], at the beginning of the year 1999, there were 169.2 km of tunnels on 1638 km of National Roads, divided into 188 tunnels. In 2005, this number has increased to 213 with a tunnel incidence of about 10.4% evaluated on the total length of the National Roads network (see Table 1.1 for further details).

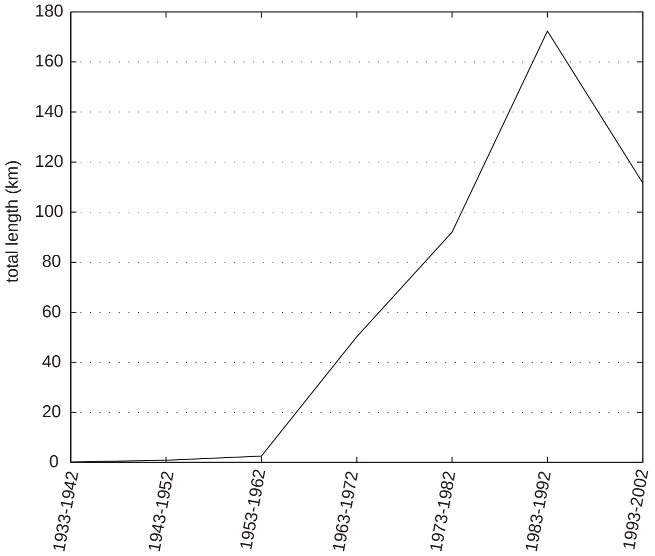


Fig. 1.3. Length of excavated tunnels per decade for the Swiss Roads network (source of data: FGU-SIA Tunnelstatistik - http://lmr.epfl.ch).

Age is an interesting feature for describing Swiss roads tunnels. As a matter of fact, National Roads began developing during the sixties, thus, today, a significant number of tunnels has already about 40 years of operation and needs maintenance to assure everyday serviceability and safety. Another specific feature directly related to age, is that all National Roads tunnels have a concrete lining. In some cases, tunnels have a reinforced concrete lining. This happens, for example, with tunnel portals that, under particular conditions, have to bear additional loads due to slopes degradation and general instabilities or when the tunnel is excavated in a swelling ground.

It is possible to identify two typical categories of National Roads tunnels:

1. Motorway tunnels which are characterised by having a quite important clearance height, high overburden and length,
2. Interconnection tunnels which are usually the direct continuation of main roads into towns and which are characterised by a great section underpassing minor hills with a low overburden.

In terms of excavation length, the longest tunnels on National Roads are:

- St. Gotthard (UR/TI) 16.6 km;
- Seelisberg (NW/UR) 9.3 km;
- San Bernardino (GR) 6.6 km.

Also without considering the longest tunnels, the mean length of National Roads tunnels is still quite important (i.e. about 1150 m), with a average of maximum overburden of about 130 m.

1.4 Conclusion

Switzerland is an important connection between the northern and the southern part of Europe, with a quite developed transportation network which needs interventions in terms of maintenance and conservation. In Switzerland, traffic tunnels excavation began developing a lot with railways construction during the second half of the XIXth century, while road tunnels began developing a lot during the sixties. Compared to other European and extra European countries, Switzerland has a remarkable number of tunnels mostly due to its mountainous topography.

Due to this richness in underground structures (i.e. about 10.4% tunnel incidence for National Roads as reported in Table 1.1) and their needs for maintenance and conservation, the Swiss Federal Roads Authority concerns itself with the elaboration of efficient conservation practices. This work aims at contributing to these reflections by analysing the main factors that influence the long term behaviour and degradation of road tunnels, focusing on the main features characterising the Swiss National Road network. After a detailed analysis of degradation problems affecting tunnels, with particular attention to road tunnels specificities (see Chapter 2), together with a comparative study of conservation techniques applied in several countries (see Chapter 3), Chapter 4 describes the whole process that lead to the Data Base creation. Then, by means of several analyses, worked out using both G.I.S. tools and multivariate statistical methods (see Chapter 5), the main factors that influence long term behaviour of road tunnels are pointed out. Finally, a specific methodology that takes into account tunnel degradation for evaluating its long term behaviour is described. Then, in the framework of the Convergence-Confinement method some examples of long term equilibrium conditions evaluations are proposed (see Chapter 6), together with application to real cases (see Chapter 7). The degradation models presented in Chapter 2 are used to describe the long term behaviour of both the rock mass and the tunnel lining. Figure 1.4 shows the structure of this work and the main contents:

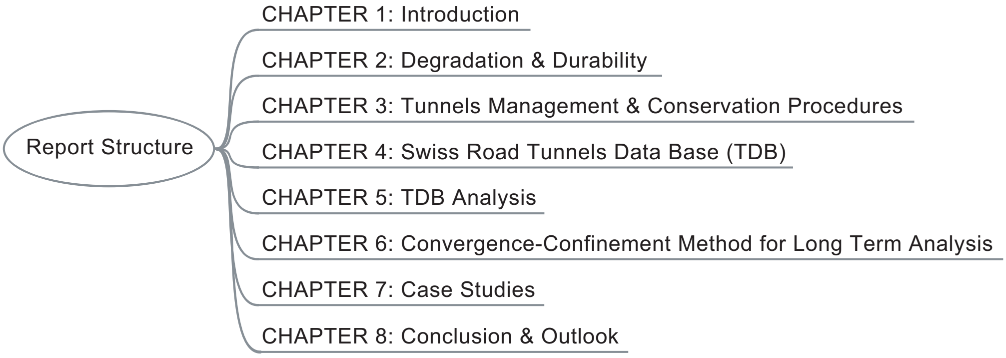


Fig. 1.4. Report structure and main contents.

2. Degradation and Durability

Durability is the capacity of a structure to last in time, performing the service for which it has been designed. During its life a tunnel may be affected by several types of problems. Incidents, can cause service life interruptions and may appear in the form of exceptional and unexpected events as earthquakes, or, in the majority of cases, fires during operation. Tunnel collapses and face instabilities due to geological uncertainties are typical of tunnel construction process. During service life, general instabilities are quite rare and, usually, tunnels show symptoms of degradation processes in the form of disorders. All these problems may weaken the tunnel until it is necessary to operate repairs or, in worse cases, a complete tunnel refurbishment.

Compared to other structures, the distinctive feature of a tunnel is the direct interaction with the rock mass. As a matter of fact, rock mass and lining characteristics may change with time and modify tunnel stability conditions in the long term. Thus, tunnel degradation can be divided into three different levels of problem [128]:

- Lining deterioration (visible surface level),
- Rock mass degradation (surrounding structure level),
- General structural instability (global behaviour).

As already seen in Chapter 1, an important feature that characterises Swiss National Roads tunnels is the concrete lining. Thus, long term lining deterioration will be described by considering only concrete pathologies. Based on a detailed literature analysis, this chapter is structured as follows:

1. Analysis of long term behaviour of road tunnels,
2. Identification of degradation mechanisms, pathologies that affect road tunnels during their service life,
3. Detailed description and selection of appropriate model for each degradation process,
4. Specification of design service life and tunnel durability.

2.1 Tunnel long term behaviour

Mainly due to the age of railway network (see also Table 1.1), the attention of several authors in the past has been focused on railway tunnel deterioration. [142] already pointed out some interesting observations about tunnel degradation with time and operation. [53] proposed several typologies of intervention for rehabilitation and repair of railway tunnels. Moreover, in 2004, the CFF (Swiss Federal Railway Company) proposed a disorders check list [32] focusing on problems that may affect railway tunnel linings.

Only recently, tunnelling associations (e.g. [30]; [72]) began to concentrate on problems typical of road tunnels. From a general point of view, road tunnels long term behaviour can be described by three main features:

1. Time evolution has on a tunnel two main effects: degradation and changes in operational requirements for assuring both tunnel safety and serviceability.
2. Though the interface between rock mass and lining is not directly accessible by means of ordinary inspections, within lining disorders it is possible to recognise symptoms typical of excavated rock mass pathologies (mainly related with rheological behaviour and ground water circulation).
3. High traffic may increase the number of car collisions and the risk of tunnel fires. Moreover, together with humidity, traffic contributes to the aggressive atmosphere within the tunnel which may cause lining deterioration and cars may transport de-icing salts and project them against tunnel walls, footway and gutter.

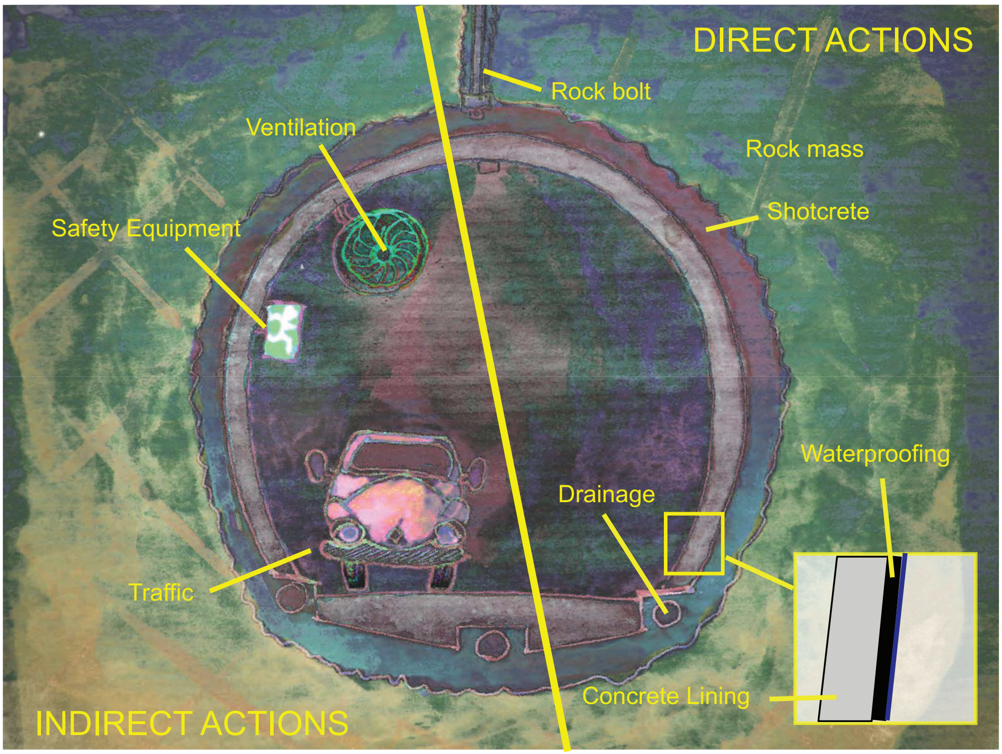


Fig. 2.1. Direct (on the right) and indirect (on the left) time actions on road tunnels.

In order to reduce tunnel degradation effects and speed, it is necessary to apply conservation procedures, as it will be described in detail in Chapter 3. It is possible to identify two type of actions (see sketch of Figure 2.1) and the procedure changes according to the process that leads to serviceability decrease, in particular:

- Direct Actions: if material quality and stability conditions change due to rock mass and/or lining deterioration, the tunnel will need maintenance, repairs and rehabilitation practices.
- Indirect Actions: if safety conditions and normal operation are no longer assured due to an inadequate conception of the whole structure, the tunnel will need structural improvements and renewal. This kind of procedure, for example, becomes necessary when the tunnel do not longer respect safety and ventilation requirements as prescribed by the norm effective at the moment of the inspection [101].

The behaviour of each structure depends on both initial conditions and evolution with time. Moreover, as shown in Figure 2.2, tunnel conditions are influenced by three main categories of factors, respectively:

1. Construction conditions (i.e. excavated rock mass, building material quality, construction method),
2. Field conditions that represent both environmental and operation factors (i.e. traffic, accidents, temperature and moisture),
3. Conservation conditions (i.e. maintenance practices, surveys).

These factors describe and refer to a specific time during tunnel life. The first class groups past features necessary to define the tunnel initial conditions. The second and the third classes represent present features and together with construction conditions allow to evaluate the evolution of tunnel condition (i.e. tunnel degradation).

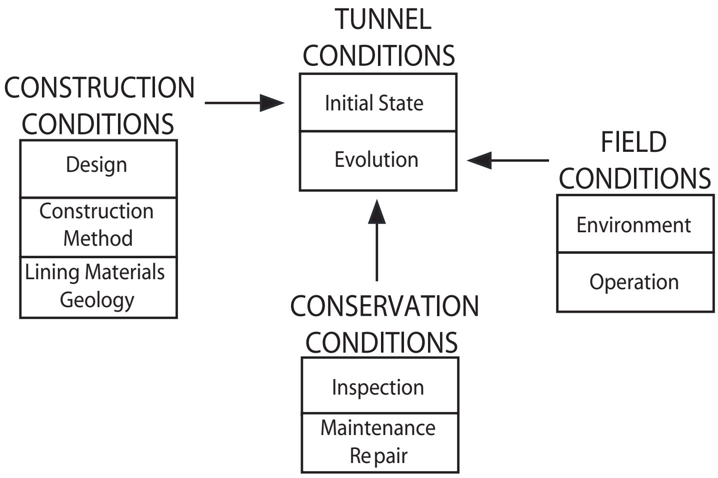


Fig. 2.2. Main classes of influence factors that determine both tunnel initial state and evolution.

Table 2.1 shows for several countries (sources codification is detailed in Chapter 3) the main causes of detected tunnel disorders. Though environmental and hydrogeological conditions change from country to country, water leakage (which mainly depends on ground water conditions, waterproofing and drainage systems) is considered as one of the principal causes for tunnels deterioration. Furthermore, ice and frost also play an important role in lining degradation. Other identified causes are tunnel overburden, surrounding ground pressure and operational conditions. In particular, operational conditions include aggressive atmosphere, which depends above all on traffic pollution and ventilation system, and use of de-icing salts with their corrosive actions.

Tab. 2.1. Principal causes of tunnel deterioration as identified by tunnel inspectors from several countries (the information sources and documents codes are detailed Chapter 3, Table 3.1 & Table 3.2).

		Environment				Operation					Construction				
Country	**Network & Document Code**	temperature	humidity precipitation	frost, ice formation, freeze & thaw cycles	biological attack (mushrooms and micro-organisms)	traffic	pollution (=corrosive atmosphere)	fires	accidents	use of de-icing salts (chemical corrosive action of chlorides)	material quality	age	surrounding ground	groundwater	construction faults
Switzerland	Road (CH1, CH2, CH3, CH4, CH5, CH6)	X	X	X	X	X	X			X	X	X	X	X	
	Rail (CH7)						X						X		
France	Road (FR1, FR3, FR5)		X				X				X	X	X	X	
	Road/Rail (FR2, FR4)			X							X	X	X	X	
Italy	Road/Rail (I2)	X	X	X		X	X			X			X	X	X
	Road (I4)		X	X							X	X	X	X	
Portugal	Rail (P1)												X	X	X
UK	Road (UK1)	X	X	X		X				X			X	X	
USA	Road/Rail (US1, US2)	X		X		X	X	X				X	X	X	
	Road (US3)		X										X	X	
Japan	Road (J1)	X	X	X							X	X	X		
	Rail (J2)		X			X				X				X	
South Africa	Road/Rail (SA1)		X	X	X	X	X							X	X

2.2 Tunnel degradation

Various chemical, physical and mechanical processes may determine exposed surfaces of both rock mass and lining to deteriorate. It is possible to distinguish between the chemical action, properly named weathering, and the physical-mechanical actions which may disturb exposed surfaces and increase the potential action of weathering factors.

Based on the process type and on its effects on both tunnel and lining it is possible to distinguish three main classes of tunnel degradation:

1. Ageing processes which group physical (mechanical) processes ruled by time dependent changes of the internal characteristics of both the lining material and the rock mass. This kind of processes may result in a reduction of the material stiffness (e.g. strain increase).
2. Weathering processes which group all chemical processes that affect the tunnel surface and/or the weak points (e.g. discontinuities). This kind of degradation is caused by the interaction of the tunnel with the surrounding environment, lining weathering, for example, is mostly influenced by operational conditions, waterproofing system and chemical composition of ground water. These processes mainly affect material strength and mechanical properties of the material. Moreover, thickness reduction can be observed for the tunnel lining.
3. Other actions, mainly mechanical (physical) as ice and frost action, car collisions and fires that may change the tunnel behaviour with time by reducing its strength or thickness (i.e. of the tunnel lining).

Both ageing and weathering, together with water and other physical-mechanical actions, are responsible for tunnel evolution, which slowly changes its serviceability, stability and safety conditions. Deterioration rate depends not only on active processes but also on which part of the tunnel is affected. For example, the interface between lining and rock mass results more exposed to ground water action, while the tunnel lining intrados can be more affected by internal humidity and operational conditions.

Starting from these considerations and based on a detailed bibliography research, Table 2.2 and Table 2.3 point out the main pathologies for both concrete lining and rock mass, by considering:

- Degradation type: ageing, weathering, other actions (mainly mechanical and physical)
- Process type: physical, chemical and/or mechanical,
- Tunnel Pathology
- Existing models for describing pathologies effects in terms of tunnel behaviour (i.e. stiffness and/or strength properties decrease),
- Potential influence factors
- Observed disorders,
- Tunnel parts affected by identified disorders.

During inspections, the perception of tunnel global conditions is mainly based on lining quality and aspect. Lining pathologies may appear in the form of several disorders depending on building material (e.g. quality and porosity), construction techniques and operation conditions. As written in Chapter 1, due to their age, all National Roads tunnels have concrete or reinforced concrete linings. Like other materials, concrete is affected by long term degradation (see Table 2.2): it usually begins on exposed surfaces, and, under water pressure, may accelerate through cracks [61]. Weathering factors, together with structural ageing contribute to modify lining characteristics. Nevertheless, also ground water pressure and rock mass behaviour may change significantly with time, as shown in Table 2.3.

Pathologies effects can be modelled by modifying strength and/or stiffness of both the lining and the rock mass of both the lining and the rock mass. Thus, starting from tunnel initial conditions it would be possible to evaluate the tunnel degradation and its actual stability condition. Moreover, by means of appropriate models it should be possible to predict tunnel long term stability conditions.

Tab. 2.2. Tunnel degradation: pathologies and observed disorders of concrete lining.

Process	**Type**	**Pathology**	**Model**	**Influence Factor(s)**	**Observed disorder**	**Where**
Ageing	Physical, (Mechanical)	Creep & Relaxation	Deformation increase Stiffness reduction	Time, Humidity, Temperature, Stress	Deformations, Cracks	Crown, Walls
		Shrinkage		Material quality	Surface skin micro-cracks	Walls
Weathering	Chemical	Calcium Leaching	Porosity increase, Strength reduction, Thickness decrease	Water circulation, Waterproofing system, Ground water chemical composition, Rock mass chemical composition, Atmosphere chemical composition, Ventilation system, Length, Traffic, Temperature, Relative humidity, Degree of saturation, De-icing salts quantity, Lining material	Concretions, Weathered surfaces	Crown, Ventilation slab, Joints
		Chlorides/De-icing salts Attack			Reinforced concrete corrosion, Oxidation, Cracks, Scaling	Footway, Gutter, Wall foundations, Walls up to 1.5 m high, Portals
		Sulphates Corrosion			Concretions, Efflorescence, Weathered surfaces, Scaling	Crown, Walls, Ventilation slab
Other Actions	Mechanical, (Physical)	Ice, Frost	Thickness decrease, Strength reduction	Temperature, Water incomes, Humidity	Weathered surfaces, Cracks, Scaling	Portals, Crown
		Accidents (car collision)		Traffic	Damaged surfaces, Cracks, Scaling	Gutter, Footway, Walls up to 1.5 m high
		Fires		Temperature, Lining porosity	Spalling, Scaling	General

Tab. 2.3. Tunnel degradation: pathologies and observed disorders of excavated rock mass.

Process	**Type**	**Pathology**	**Model**	**Influence Factor(s)**	**Observed disorder**	**Where**
Ageing	Physical, (Mechanical)	Creep (secondary creep)	Deformation increase Stiffness reduction	Time, Rock mass type, Ground water presence, In situ stress	Cracks, Deformations	Crown, Walls
		Squeezing (tertiary creep without damage)				
	Physical / (Chemical)	Swelling	Deformation increase Stiffness reduction	Rock mass type, Ground water presence, In situ stress	Invert heave	Wall found., Invert
Weathering	Chemical	Soluble minerals leaching	Strength reduction, (plastic radius increase), Changes in water pressure around excavation	Ground water chemical composition, Rock mass type and mineralogical composition, Water circulation around excavation (i.e. function of waterproofing and drainage systems), Excavation method,	Weathered surfaces, All disorders already mentioned for lining weathering, Drainage calcareous concretion	Crown, Walls, Drainage system
Other Actions	Mechanical, (Physical)	Consolidation	Dissipation of pore water pressure	Ground mass porosity, Water circulation around excavation (i.e. function of waterproofing and drainage systems),	Settlements, Deformations around the excavation	Crown, Walls, Wall found.
		Fines leaching and transport	Strength reduction (cohesion decrease), Changes in water pressure around excavation	Ground mass lithology, Water circulation around excavation (i.e. function of waterproofing and drainage systems),	Filling of drainage system	Drainage system
		General slope instability, Block falls	Strength reduction (mechanical properties decrease)	Rock mass stability conditions	Cracks, Structural collapse	Portals

As it is possible to see by comparing the two tables, quite often, the same disorder can be the consequence of several pathologies. It is, thus, important to follow evolution with time in order to identify the main cause. For example, in the case of lining cracking, according to several authors [142]; [7] and [30] it is possible to recognise different kinds of cracks:

- Transversal cracks which are normally closed and mainly appear on tunnel walls. In particular, when cracks are perpendicular to the tunnel axis they may be a symptom of changes in the external loads (e.g. due to the weathering of exposed slopes). For example, walls cracks may indicate changes in the external ground pressure, while cracks along the bottom of the walls may indicate foundations settlement.

- Longitudinal cracks which are normally opened (max. 2-3 mm) and affect the tunnel crown and the side walls (see Figure 2.3). They may be caused by inadequate load-bearing capacity of the tunnel section (i.e. due to both insufficient lining or excessive loads). In particular, when cracks open along the spring line they may be the consequence of voids and cavities behind the lining (which reduce the lining bearing capacity).
- Diagonal cracks which may be caused by vertical movements of the tunnel.
- Small fissures which are usually symptom of ice and frost actions, corrosion, water inflow.
- Surface skin cracking or micro-cracks which mainly affect tunnel walls and can be systematic especially in recent tunnel lining. This kind of fissures is the direct consequence of the hydration process, but, sometimes, it may be caused by low material quality and construction defects.

While the first three types of cracks may influence the tunnel stability conditions by reducing the lining strength, usually, the third and the fourth types do not change the tunnel stability conditions.

Cracks and fissures may also be the symptom of Alkali-Aggregate Reaction (AAR). As a matter of fact, in presence of water and in an alkaline cement matrix, some kinds of aggregates react and form a gel. This process, also called “concrete cancer” [87], is an expansive reaction. If the volume increase is constrained (e.g. in case of reinforced concrete) it causes cracks formation. The crack pattern and the severity of this pathology mainly depends on the quantity of water and on the concrete lining conditions. Actually, when a concrete structure is already affected by surface disorders humidity of the atmosphere may accelerate the reaction often AAR is associated to other kinds of deterioration, as, for example, disorders caused by de-icing salts frost attack and chlorides corrosion.

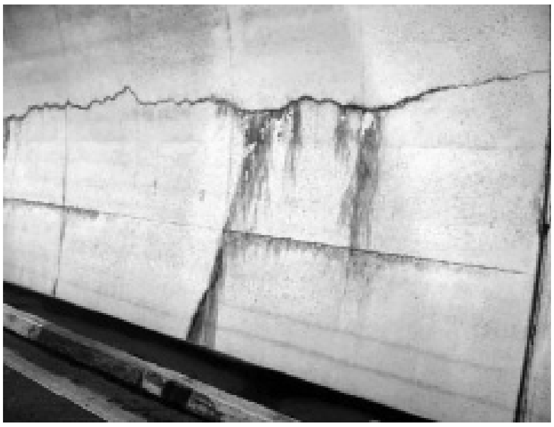


Fig. 2.3. Longitudinal wall crack and water income (courtesy of Canton Ticino).

2.2.1 Water action

[142] already identified water seeping through the tunnel walls as “the most common cause of future troubles”. As shown in Table 2.1, water is considered by the majority of tunnel inspectors one of the major causes of tunnel degradation. A a matter of fact, the combined action of water and frost may cause tunnel degradation. The rock mass evolutive behaviour can be highly influenced by ground water as it happens, for example, to argillaceous rock masses as shales, clays and argillitic sandstones when saturated. Moreover, according to several authors [142]; [69]; [20] water may reduce mechanical properties of the excavated rock mass by dissolution of chemical components.

A particular feature of underground structures is that water and humidity may come from both the rock mass (i.e. ground water) and the environment (i.e. condensation water and moisture): usually, the tunnel atmosphere is characterised by having between 40% up to 95% of relative humidity (RH) depending on ventilation, waterproofing system and season. The literature [22]; [129] considers relative humidity (RH) of about 50 up to 80% a potential aggressive agent for concrete structures. Moreover, ground water with its weathering potential may reduce both quality and mechanical properties (e.g. bearing capacity) of the excavated rock mass and, mainly depending on waterproofing system, of the lining. Ground water contributes to tunnel deterioration not only by transporting aggressive substances, but also by changing external load conditions (mechanical action, i.e. water pressure). As a matter of fact, tunnels below ground water table are exposed to the additional pore water pressure. Depending on the possibility of a water leakage through the tunnel lining and on pore water pressure redistribution after tunnel construction, ground water pressure may change with time (i.e. consolidation). Thus, as described in [69]; [58]; [20], effective stress effects which may not be significant during tunnel construction, should be considered after the final lining completion for evaluating long term tunnel behaviour. As already reported in Table 2.2 and Table 2.3, there are three different types of water action:

1. External action: it can be the degradation due to frost and ice, the erosive action of freezing-thawing cycles at tunnel intrados; the weakening action of ground water at the interface between excavated rock mass and lining.
2. Structural action: the water can be charged in aggressive substances that affect concrete (e.g. Sulphates) and/or steel reinforcements (e.g. Chlorides). Structural instability, collapses and subsidence, may be caused also by rising water table or by changing pore water pressure around the tunnel (e.g. due to ground mass consolidation).
3. Functional action: substantially, this represents drainage system pathologies (e.g salt concretions, fines particles deposits). As a matter of fact, fine particles transported by ground water and/or calcareous concretions, may obstruct drains and create unexpected water pressures around the tunnel which may result in lining failure.

As represented in Figure 2.4, water action is mainly chemical. When the tunnel lining is not protected by a waterproofing system, ground water, charged in salts can be very aggressive for concrete [124]; [30]; [129]; [62]. By seeping through the lining, ground water dissolves soluble cementitious constituents and this process may result in cracks opening. The consequent concrete porosity increase accelerates the degradation. This process, also called “wick action” [121], is typical of concrete lining dried by tunnel ventilation system. According to [30] this process has remarkable effects with materials at 60% of relative humidity and/or in the case of old concrete. The ventilation within the tunnel dries and cause the unsaturation of the tunnel walls and the oxidation front tends to move inward from the tunnel intrados. This process results in mechanical properties decrease and in a global weakening of the affected area.

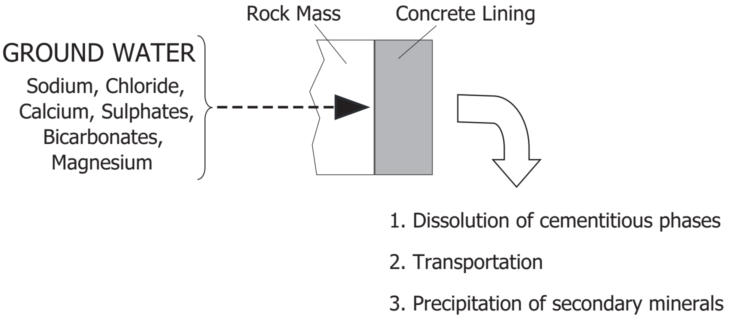


Fig. 2.4. Ground water chemical action on a concrete tunnel lining.

As ground water can be very aggressive for concrete linings (see Figure 2.5), it is important to know its chemical composition. As reported in [16], a study made by Swiss Federal Institute of Technology in Zurich (ETHZ) in 1994 identifies four types of corrosion (for concrete and reinforced concrete structures), based on ground water chemical composition (further details in Chapter 5):

I. water with high contents in SO_4^-, Na^+, Mg^{2+}, Ca^{2+} and Cl^-;

II. salt water with medium contents in Na^+, Mg^{2+}, Ca^{2+}, Cl^- and SO_4^-;

III. water with HCO_3^- with high contents in CO_2, Na^+, Mg^{2+}, Ca^{2+}, Cl^- and SO_4^-;

IV. deep water with high contents in Ca^{2+}, Cl^- and SO_4^-.

Fig. 2.5. On the left: water inflow; on the right: efflorescence (courtesy of Canton Ticino and État de Vaud).

2.2.2 Concrete lining degradation

Since their construction, tunnel linings can be affected by typical concrete pathologies mainly due to material quality and construction defects as, for example:

- AAR (Alkali-aggregate reaction) generated by reactive aggregate in the concrete ingredients [129]
- Honeycomb, normally caused by inadequate vibration during concreting process [30].

Then, during, during service life, tunnel linings start to show typical symptoms of degradation processes (e.g. ageing, weathering or other actions).

Ageing

As summarised in Table 2.2. ageing groups all pathologies as creep and shrinkage which depend (mainly) on intrinsic material properties as quality and age. Several authors [14] tried to study and model this phenomenon in the past. Benboudjema et al. in [122] identify two types of creep for concrete structures:

1. short term creep, that develops just few days after concreting,
2. long term creep, a very slow process that depends only on time, (without any apparently asymptotic stabilisation).

For the long term creep, they show by means of laboratory tests that the last in time is directly proportional to the square of the length. For instance, it takes approximately 10 years for a concrete sample of about 16 cm, (80-100% water content; 50% of environmental humidity) to complete the process.

Anyway, as reported by [94], about 75% of the 20-year creep occurs in only 1 year. Moreover, beyond 20 years the creep increase for normal concrete structures is negligible.

The literature proposes several models (e.g. [122]; [28]) for predicting the long term strain caused by creep. Some of these models are used by norms (e.g. Eurocode 2 is based on

CEB-FIP Model Code 1990) for evaluating the influence of ageing on concrete structures behaviour. The elastic strain ε_t increases on the long term can be expressed as follows:

$$\varepsilon_t = \varepsilon_{t_0}^{el} + \varepsilon^c \tag{2.1}$$

where $\varepsilon_{t_0}^{el}$ is the instantaneous elastic strain evaluated at the loading time t_0 and ε^c is the creep strain evaluated at time t. For a concrete element, ε^c can be evaluated as prescribed by Model Code 1990 [28]:

$$\varepsilon^c = \frac{\sigma_{t0}}{E_{t0}} \cdot \bar{\varphi}(t, t_0) \tag{2.2}$$

where σ_{t0} is the applied stress and E_{t0} the Young's Modulus at time t_0, and $\bar{\varphi}(t, t_0)$ is the creep coefficient

$$\bar{\varphi}(t, t_0) = \varphi_0 \cdot \beta_E \cdot \beta_{t_0} \cdot \beta_{RH} \cdot \beta_\sigma \cdot \beta_t \tag{2.3}$$

with:

- φ_0 function of the concrete quality;
- β_E coefficient for evaluating the evolution of the Young's Modulus with time;
- $\beta_{t_{0(0)}}$ function of the loading time;
- β_{RH} function of both relative humidity and temperature;
- β_σ depending on the stress level (note that this factor can be considered only if $(0.4 \cdot f_c) \le \sigma \le (0.6 \cdot f_c)$, with f_c compressive strength of concrete aged of 28 days);
- β_t coefficient for evaluating the evolution of creep with time.

Thus, concrete structures creep depends on concrete quality, age, loading time, stresses, temperature and humidity. By adapting boundary conditions to tunnel operation and environment (e.g. ventilation system, humidity, temperature and loads), it is possible to evaluate creep also for a tunnel lining.

Weathering

As it is summarised in Table 2.2, another important degradation process typical of concrete structures is chemical deterioration. Tunnel lining may interact with the surrounding environment by exchanging ions [62], this causes chemical weathering reactions and changes concrete mechanical properties. Weathering groups all pathologies caused by interaction of exposed surfaces (i.e. also discontinuities) with “external factors” (e.g. ground water, moisture, chemical components of the rock mass, traffic pollution). The interaction between the tunnel lining and its surrounding environment may result quite often in a concrete deterioration. In particular, corrosive soils may affect concrete lining through water circulation which is essential for chemical exchanges. Ground water action has been already described in Section 2.2.1. Ions transported by water and/or gases due to traffic pollution may be quite aggressive for road tunnels concrete lining. According to [30], among the corrosive gases due to traffic pollution, carbon dioxide (CO_2) and sulphur dioxide (SO_2) are the most aggressive for plain concrete. While reinforced concrete lining (e.g. tunnels portals and some tunnels constructed at the beginning of the 1970s), are exposed to carbonation and steel bars corrosion mainly caused by de-icing salts used during winter season. Moreover, this process may accelerate due to corrosive electric currents generated by high oxygen content and ground porosity around the excavation, together with a small grain size of ground mass.

Corrosive soils were already described by [142]: low carbonic acid content, high degree of acidity, high salts and moisture content, good conductivity. The norm [134], (SN EN 206-1, 2000) identifies several classes of concrete structures exposure to environmental actions. For each class the norm selects parameters for defining critical exposure. Thus, by analysing both tunnel environmental and operational conditions (e.g. traffic pollution and use of de-icing salts), it is possible to evaluate the lining exposure class for the lining. In particular, as it is summarised in Table 2.4, the sixth class takes into consideration corrosive action of natural soils, ground and surface water. Within this class, the norm distinguishes three exposure subclasses by defining the main aggressive factor and its concentration:

I. XA1 which represents the less aggressive environment

II. XA2,

III. XA3 which corresponds to the most aggressive one.

Tab. 2.4. Limit values of exposure to chemical attacks (6th exposure class) due to ground water and soil composition [134], (SN EN 206-1, 2000). For each class the norm identifies the principal aggressive factor/agent and its concentration (i.e. maximum and minimum values)

		Exposure Class		
	Aggressive Factor	**XA1 = Weak**	**XA2 = Medium**	**XA3 = Strong**
Ground water and surface water (mg/l)	SO_4^{2-}	200 to 600	600 to 3'000	3'000 to 6'000
	pH	6.5 to 5.5	5.5 to 4.5	4.5 to 4.0
	CO_2	15 to 40	40 to 100	>100 (saturation)
	NH_4^+	15 to 30	30 to 60	60 to 100
	Mg^{2+}	300 to 1'000	1'000 to 3'000	>3'000 (saturation)
Surrounding rock mass and soil	SO_4^{2-} (mg/kg total)	2'000 to 3'000	3'000 to 12'000	12'000 to 24'000
	Acidity (ml/kg)	>200	-	-

CALCIUM LEACHING. Within concrete chemical deterioration processes, one of the most important is Calcium leaching (see Figure 2.6). The dissolution of soluble cement constituents due to water leakage is described by reactions in Equation 2.4 and Equation 2.5 [96], where carbon dioxide is a result of exhausted gases from traffic free (i.e. CO_2) or linked with water (i.e. HCO_3):

$$Ca(OH)_2 + CO_2 \leftrightarrow CaCO_3 + H_2O \tag{2.4}$$

$$CaCO_3 + CO_2 + H_2O \leftrightarrow Ca(HCO_3)_2 \tag{2.5}$$

Even if this chemical process in itself is not very dangerous for the whole structure, this chemical attack is not only a simple aesthetic deterioration. As reported by Adenot, 1992 and Kamali, 2003 [129], the leaching process follows the Fick's diffusion law (Equation 2.6) where the flux, J, is a function of the Diffusivity, D, and the concentration gradient):

$$J = -\left(D \cdot \frac{\partial}{\partial x}C(x, t)\right) \tag{2.6}$$

The process speed may, however, accelerate due to the presence of Chlorides or water temperature. [152] analyse the behaviour of structures of 34 up to 104 years old and state that the degradation thickness due to calcium leaching can reach a maximum value of 100 mm in 100 years. Normally, the thickness of the degraded zone, X_d [mm], is proportional to the square root of time, t [days], as expressed by Equation 2.7, where a is a material constant. For example, for a cement paste, a depends on the water to cement ratio (W/C).

$$X_d = a \cdot (\sqrt{t}) \tag{2.7}$$

[25] and [96] show through laboratory tests that the decrease in mechanical properties is directly proportional to the degraded area:

$$\delta A_d = \frac{A_d}{A_0} \tag{2.8}$$

where

- A_d is the degraded area,
- A_0 is the testing samples section.

Equation 2.9 and Equation 2.10 show, respectively, the evolution of Young's Modulus (E) and compressive strength (σ_c) according to [96]:

$$\frac{\delta E}{E_0} = \frac{E_0 - E}{E_0} = k_m \times \delta A_d \tag{2.9}$$

$$\frac{\delta f_c}{f_{c0}} = \frac{f_{c0} - f_c}{f_{c0}} = k_r \times \delta A_d \tag{2.10}$$

where E_0 and f_{c0} represent the Young's Modulus and the compressive strength of the sound material, $k_m = 0.66$ and $k_r = 0.76$.

[79] describe the process using a coupled chemical-mechanical model. Calcium leaching from the concrete increases the porosity and, consequently, the permeability of the lining itself. The whole process reduces both stiffness and strength of the concrete lining. The lining microfissuration (i.e. damage) may evolve into cracks formation (see Figure 2.6 right). This accelerates the exchanges between the tunnel and the environment and increases the degradation speed. Equation 2.11 shows the constitutive law that describes the damage evolution due to concrete chemical degradation according to [86]:

$$\sigma = (1 - D) \cdot (1 - V) \cdot E_0 \cdot \varepsilon \tag{2.11}$$

where

- D is the mechanical damage variable as described by [90]
- V, the chemical damage variable, is a function of Calcium ions concentration in the pore solution.

Recently, mechanical properties reduction due to increasing porosity caused by calcium leaching, has been demonstrated also by [62].

Fig. 2.6. Calcium leaching (courtesy of État de Vaud). On the right an enlargement.

SULPHATES CORROSION. Sulphates, mostly conveyed by ground water inflow or, especially in the past times, originated by exhausted gases, are particularly aggressive with normal concretes. As a matter of fact, together with evaporites, also clayey soils and crystalline rocks rich in iron sulphide, once exposed to the air, form with the seeping ground water a sulphate solution that may affect concrete lining [142]; [68]. Sulphates may lead to significant mechanical damage (i.e. cracking) mainly due to differential internal expansions. Problems due to sulphates in tunnels have been observed and studied by

[51] and [52]; [150]; [124]; [30]; [119]. The chemical reaction of sulphates and concrete may generate thaumasite in the cement paste (see Equation 2.12) [129]. This reaction depends on both construction and environmental conditions (e.g. geology, construction method, concrete type, thickness and tunnel atmosphere).

$$\{Ca_6[Si(OH)_6]_2 \times 24H_2O\} \times [(CO_3)_2 \times (SO_4)_2] \tag{2.12}$$

At present, it doesn't exist any model for describing mechanical properties decrease of a concrete structure affected by sulphates corrosion. Anyway, according to the exposure class to sulphates attack (i.e. quantity of sulphates), the norms prescribe the type of concrete to use. Usually, this kind of concrete is made using a cement paste type HS with very low content of C_3A (i.e. less than 3%) which may show some problems due to a lower resistance to Chlorides attack. [124] affirms that the alteration of more than 20% of the cement paste into thaumasite cause the complete loss of cohesion of the concrete. Moreover, experimental tests made by [65] on Portland limestone cements exposed to sulphate attack show that, after only one year, the compressive strength has significantly decreased (i.e. up to 75%, for samples exposed to $MgSO_4$ solution at 5°C). Visible symptoms of sulphates attack are efflorescence and weathered surfaces. Then, as it is a time dependent process, the alteration progresses from the surface towards the interior and results in softening of the whole material. Sometimes, as the chemical reaction of Equation 2.12 induces a volume increase, tensile stresses originate and may cause a loss in lining bearing capacity and irreversible damages to tunnel.

DE-ICING SALTS EFFECTS & CHLORIDES CORROSION. As it happens for sulphates, ground water may be also charged in Chlorides (some rock formation have a quite high content of Chlorides, as for example Lias and Opalinus Clay). However, generally, chloride ions concentration in pore water is mainly due to the use of de-icing salts. [148] investigate the effects of several types of de-icing salts on concrete structures and demonstrate, by means of laboratory tests, that chloride solutions (i.e. $CaCl_2$ and $NaCl$) caused the most damage. As reported by several authors [74]; [22]; [129]; [33]; [148] the use of de-icing salts may have several consequences:

- concrete saturation and risk of frost damage increase,
- decrease of the freezing point of the pore solution which, as the salt concentration decreases with the distance from the exposed surface, leads to significant hydraulic pressures and concrete scaling due to ice formation below the surface
- chemical deterioration due to interaction between de-icing chemicals and concrete material.

Thus, successive applications of de-icing salts cause scaling, pitting, spalling and flaking of concrete surfaces. As it happens for Calcium leaching, chemical deterioration accelerates with permeability increase due to micro and macro cracking of concrete. During cold winter and with a frequent use of de-icing salts, this cycle can repeat and concrete structures degradation accelerates reducing lining thickness (see Figure 2.8 right). Based on prescriptions for concrete structures [134] (SN EN 206-1, 2000), it is possible to identify for each tunnel part the exposure class to de-icing salts projection and Chlorides corrosion in case of reinforced concrete. Figure 2.7 shows different exposure classes in a tunnel considering the effects of rolling traffic. In case of reinforced concrete it is possible to identify two main zones:

- XD1 - Mist: tunnel crown (or, if case, ventilation slab),
- XD3 - Splash: side walls up to 1.5 m height.

In case of plain concrete the major action is represented by de-icing salts frost attack and two exposure classes can be identified:

- XF2 - Mist: tunnel crown (or, if case, ventilation slab) slight saturated with de-icing salts,
- XF4 - Splash: side walls up to 1.5 m height strong saturated with salts.

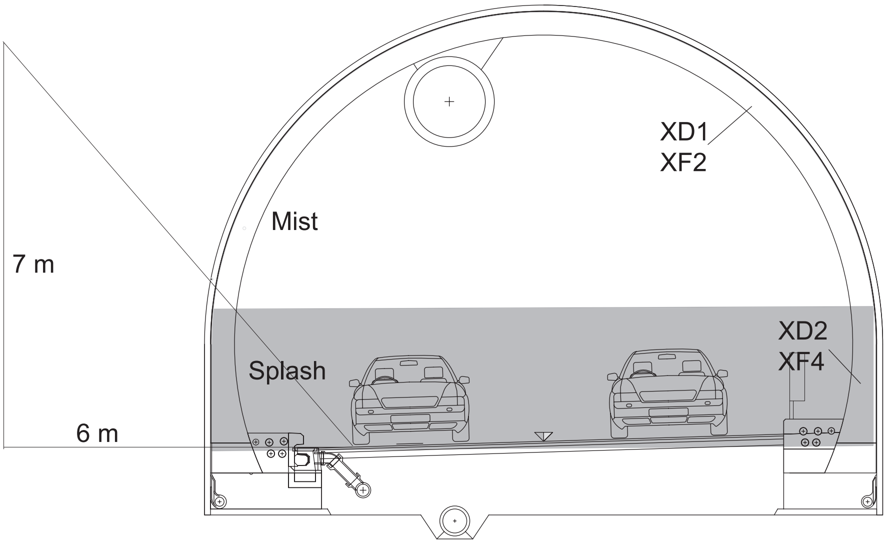


Fig. 2.7. Tunnel lining exposure zones to de-icing salts projection. Rolling traffic may project Chlorides up to 1.5 m height as evaluated by using distances and exposure classes proposed for bridges in [134] (SN EN 206-1, 2000), and a typical section of a highway tunnel (see also [33]).

As Chloride ions may penetrate into the concrete structure, when the tunnel has a reinforced concrete lining, the major problem is the corrosion of steel bars (see Figure 2.8 left). According to several authors, as reported by [59]; [22]; [129], this chemical reaction may occur when the ratio between the free chloride ions content and the hydroxyl group content lies between 0.3 and 1.0. In this reaction, pore water (i.e. concrete moisture) and oxygen are catalysts. As steel corrosion by Chlorides is an expansive reaction and generates tensile stresses in the concrete, thus when these stresses exceed the lining tensile strength, the concrete begins to spall. With a simplistic/deterministic approach, Chloride penetration with time can be evaluated by using Chloride profiles as given by Equation 2.13 [22]; [129]; [33]. Knowing the free Chlorides content, $C_{Cl^-}^{free}$, allows determining the initiation of the corrosion process for reinforced concrete (i.e. limit values as suggested by [SIA 262, 2003 (SN EN 206-1, 2000)]).

$$C_{Cl^-}^{free}(x,t) = 0.6 \cdot C_{Cl^-}^{0} \cdot \left(1 - erf\left(\frac{x}{2 \cdot \sqrt{D_c \cdot t}}\right)\right) \tag{2.13}$$

with

- $C_{Cl^-}^{0}$ exposed surface Chloride content [%];
- x depth from concrete surface [m];
- D_c diffusion coefficient [m^2/s];
- t time [s];
- erf error function (from solving Fick's second law of diffusion): $erf(y) = \frac{2}{\sqrt{\pi}} \cdot \int_0^y e^{-z^2} dz$.

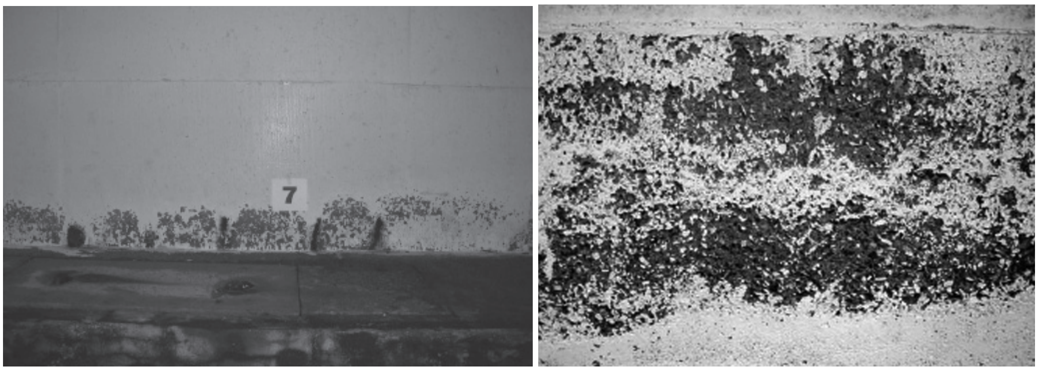


Fig. 2.8. Chlorides effects. On the left: corrosion of reinforcements in wall foundations, footway and gutter; on the right: side wall concrete damage by de-icing salts (courtesy of Canton Ticino).

In case of plain concrete structures, as shown in Figure 2.8 right, the main problem is concrete scaling due to frost de-icing salts attack at the lining intrados. [74], in his PhD thesis, proposed a qualitative model for describing frost de-icing salts damage. According to this model, three main factors determine this kind of concrete lining deterioration:

1. high concrete lining saturation degree (i.e. about 90%),
2. low temperature,
3. de-icing salts (i.e. $NaCl$ concentration of about 2-4%).

Thus, lining scaling due to de-icing salts attack can be described as the intersection of three main conditions as represented in Figure 2.9.

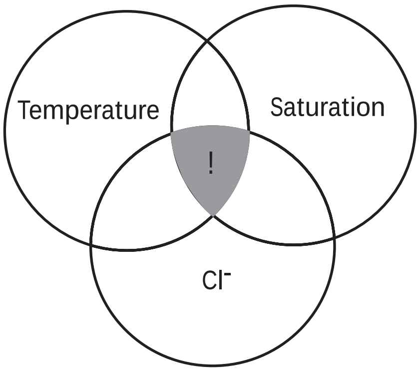


Fig. 2.9. The intersection of the three circles identifies the main conditions for frost de-icing salts attack to concrete lining (after [22]).

Other actions

There are other physical and mechanical actions that may cause lining degradation (e.g. cracking and spalling) but for which it is quite difficult to find appropriate models for describing their effects on concrete lining properties. For example, frozen water (from both tunnel atmosphere and rock mass) and repeated freezing-thawing cycles may induce tensile stresses in the lining with consequent cracking and/or spalling. These disorders can be quite important next to the portals and in short tunnels, when the tunnel lining is directly exposed to external weather. Also operational conditions may accelerate the lining degradation. For example, impacts caused by car collisions may cause severe degradation especially of footway, gutter and side walls. Moreover, as demonstrated by [97], concrete heating caused by fires during tunnel service life may significantly reduce concrete stiffness and strength, partly due to chemical changes and moisture transport in the cement paste and partly to damage.

2.2.3 Rock mass degradation

Though if only the lining is object of regular inspections, observed disorders may be a symptom of rock mass pathologies. According to [17] and a characterisation proposed by [6], rock masses may show three types of delayed behaviour:

1. creep and relaxation (damage and increase of microcracks number),
2. consolidation (changes in water flow and pressure in pores and cracks),
3. swelling (ground water presence - gypsum, rock salt, marl, claystone and chalk).

Moreover, [6] identifies two main types of rock mass weathering:

- hydro thermal alteration, caused by water flow and external aggressive agents;
- internal weathering, caused by mineralogical changes in the matrix.

Thus, excavated rock/ground mass degradation process may depend on several factors:

- excavation age and rock mass rheological behaviour (stress-strain relationship, ground water flow conditions, progressive loss of strength),
- climatic/environmental influences (e.g. moisture, temperature),
- chemical influences (e.g. ground water chemical composition, lithology, porosity, discontinuities) and unstable substances (e.g. texture size of minerals and specific surface).

A recent study made by INERIS (see Table 2.5, [85]) proposes a classification of degradation potential of several types of rock formations under physical and chemical actions.

Tab. 2.5. Rock formations degradation potential under physical and chemical actions: 1= none, 2 = low, 3 = medium, 4 = high (after [85]).

	Physical action			**Chemical action**	
Geology	**Thermal (Temperature)**	**Hydraulic (Water circulation & pressure)**	**Mechanical (Loads)**	**Leaching**	**Oxidation**
Gypsum	3	4	4	4	3
Chalk	2	4	4	4	3
Sandstone	2	4	4	3	3
Limestone	2	4	3	4	3
Marl/Mudstone	2	4	4	3	4
Clay	2	4	3	2	4
Carbon	4	3	4	1	2
Gneiss	4	2	3	1	2
Granite	4	2	3	1	2
Basalt	4	2	3	1	2
Marble	3	2	3	1	2

Weathering

According to [20] the interaction between rock mass exposed surfaces and ground water is similar to conventional building materials weathering (e.g. concrete, Section 2.2.2). During the past years, as reported in [64], several authors studied the chemical influence of ground water on the mechanical behaviour of crystalline rock masses. The presence of water reduce the ultimate compressive strength of rocks, as shown in Figure 2.10. The amount of this reduction depends on the pH of ground water (more details in Chapter 5).

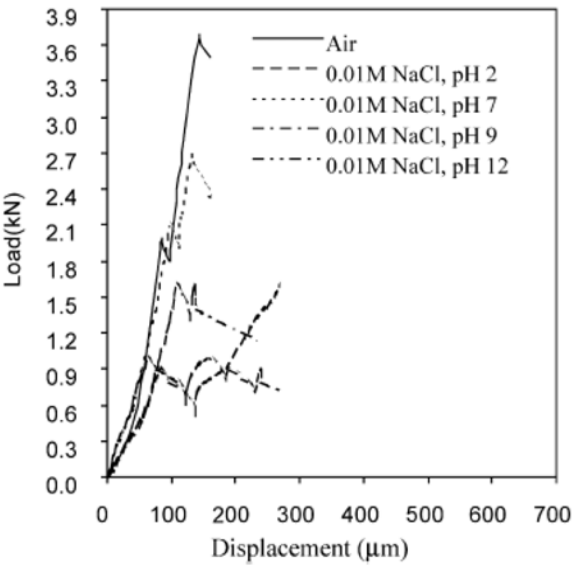


Fig. 2.10. Weathering effects of pH of ground water in crystalline rocks, in [64].

Compared to other geological formations, sedimentary rocks are particularly sensible to water action (both physical and chemical) and to oxidation. This is due mainly to their porosity, which allows air and water flows through them. [110] proposed, for unlined tunnels, a relation between depth of weathered face and tunnel age, which is very similar to Equation 2.7.

$$X_d = R_{oxi} \cdot (\sqrt{t}) \qquad (2.14)$$

In this case X_d is express in cm, *a* is called R_{oxi} and depends on both the oxygen diffusion coefficient and chemical composition of the rock mass and *t* is the time, since the tunnel construction, expressed in years. Ground water flow through rock masses may cause both dissolution of soluble components and precipitation of secondary minerals.

As previously seen with concrete lining, weathering decreases rock mass mechanical properties by increasing its porosity. Based on uniaxial compression test performed by several authors [80] affirmed that the long term compressive strength is about the 70-90% of the short term value for dry and partially saturated rocks, while it falls below the 60% for fully saturated rocks and up to zero for clays. For what concerns the Young's Modulus he assumed that on the long term it may attain the 60% of his short term value.

Thus, loss of confinement of excavated rock mass continues on the long term and increases the rock mass weight on support. For instance, weathering of exposed slopes may result in additional loads, especially above tunnel portals.

The influence of rock mass deterioration on the long term loading conditions of the crown has been already discussed for shallow tunnels in [24]. The decrease in quality and mechanical properties of the rock mass, under gravity conditions, results in an additional load on the lining. This may cause lining cracking and, in worse cases, tunnel stability conditions can reduce up to lining failure and tunnel collapse, as described also in [142]; [30]. [143] defined the rock load as the height of the mass of rock which tends to drop out of the roof (i.e. the damaged zone around an excavated tunnel). Moreover, he determined the size and the shape of this zone depend on the geotechnical conditions of the rock mass. Quite often, as reported in [114], the broken zone is associated to the damaged zone around the excavated tunnels. The shape of this zone and its extension depends on the plastic radius R_{pl}, and, thus, on the rock mass response to the tunnel construction. [111] already suggested to increase the load on the tunnel crown by considering the weight of the plastic zone:

$$\Delta p_{crown} = \gamma_{rm} \cdot (R_{pl} - R) \tag{2.15}$$

with

- γ_{rm} unit weight of the rock mass above tunnel crown,
- R tunnel radius and R_{pl} plastic radius.

[130] shown by means of theoretical and parametric sensitive analyses that the fraction of the overburden pressure supported by the tunnel lining decrease as depth increase. According to his results, shallow tunnels are frequently constructed in creeping material (loose ground) and should be designed for supporting the whole overburden while deep tunnels usually are constructed for bearing up to 30% of the overburden pressure. Thus, since the correction proposed by Pacher may appear too conservative, it is possible to use solutions proposed by [24] for low overburden tunnels. According to this solution, the pressure on the roof of the tunnel can be lowered by considering also the stabilising contribution of the rock mass (“arch action”):

$$\Delta p_{crown} = \frac{\gamma_{rm} \cdot R_p}{K_p - 2} \cdot \left[\frac{R}{R_p} - \left(\frac{R}{R_{pl}}\right)^{K_p - 1}\right] - \frac{\sigma_c}{K_p - 1} \cdot \left[1 - \left(\frac{R}{R_p}\right)^{K_p - 1}\right] \tag{2.16}$$

if $K_{psi} \neq 2$, and

$$\Delta p_{crown} = \gamma_{rm} \cdot R \cdot \ln\frac{R_p}{R} - \sigma_c \cdot \left(1 - \frac{R}{R_p}\right) \tag{2.17}$$

if $K_{psi} = 2$,

where $\sigma_c = \frac{2 \cdot c \cdot \cos\varphi}{1 - \sin\varphi}$ is the compressive strength of the rock, $K_p = \frac{1 + \sin\varphi}{1 - \sin\varphi}$ and $K_{psi} = \frac{1 + \sin\psi}{1 - \sin\psi}$, with c cohesion and φ friction angle according the Mohr-Coulomb criterion, and ψ dilatancy angle.

Ageing

On the long term, what differences one type of rock mass from another is the ageing speed. As a matter of fact, after tunnel excavation the rock mass may continue to creep. Creep is the attitude of the rock mass to deform with time under constant stress. Time dependent behaviour of rock masses is represented by strain increase [34]. As reported by [73], creep has been studied since the 1930s by Griggs. The strain depends mainly on the deviatoric stress which rules also creep velocity and type. If the material is not dilatant, the volume remains constant during deformation. Moreover, it is possible to distinguish three different creeps:

1. transient creep (primary creep) characterised by a deformation velocity decrease,
2. steady state creep (secondary creep) - viscosity with a constant deformation velocity,
3. accelerating creep (tertiary creep - squeezing) characterised by deformation velocity increase until failure.

Secondary and tertiary creeps are solid skeleton ageing processes. They depend on tunnel depth, rock mass strength and original stress field (i.e. before tunnel excavation). The stress state can produce an evolutive damage ultimately ending in a failure after a time interval.

Creep symptom is side wall convergence. Time-dependent displacements of the excavated rock mass around the tunnel are essentially associated with creep caused by exceeding a minimum shear stress and yielding of the rock mass around the excavation.

Excavated rock mass creep can be modelled by a total strain increase, ε (see also Table 2.3), due to a viscoplastic strain, ε_{vp}:

$$\varepsilon = \varepsilon_{el} + \varepsilon_{vp} \tag{2.18}$$

Usually, the main assumption for modelling rock mass delayed convergences is to separate time and stress effects as proposed by [139]. Thus, the time-dependent strain of Equation 2.18 becomes:

$$\varepsilon_{vp} = g(\sigma_{ij}) \cdot f(t) \tag{2.19}$$

As proposed by [17]; [18] it is possible to evaluate the viscoplastic strain by using the viscoplastic strain rate $\dot{\varepsilon}_{vp}$ as defined by Lemaitre:

$$\dot{\varepsilon}_{vp} = A \cdot \left(\frac{q - \sigma_s}{F_0}\right)^n \cdot (\varepsilon_{vp})^m \tag{2.20}$$

where

- q is the deviatoric stress in the rock mass, expressed in (MPa),
- F_0 = 1 MPa is a reference stress,
- n a constant that depends on the deformation mechanism (note $n > 1$),
- m a constant that respect the following condition: $(1 - n) < m < 0$,
- σ_s a limit stress beyond which the delayed behaviour starts,
- A expressed in (s^{-1}) is the viscosity parameter, usually described by Arrhenius's law:

$$A = A_0 \cdot e^{-\frac{\Delta G_0}{R \cdot T}} \tag{2.21}$$

where A_0 is a material constant, $R = 8.13 \text{ in } \left(\frac{J}{\text{mol} \cdot °\text{K}}\right)$ is the universal gas constant, T Is the absolute temperature (in °K), ΔG_0 is the activation energy of thermic reaction, in $J \cdot \text{mol}^{-1}$.

For the deviatoric stress power n, [45] suggest to use values between 3 and 6 in case of deformation of the crystalline structure (i.e. dislocation), and between 2.3 and 4.7 in case of glide (i.e. grain boundary sliding). Table 2.6 summarises values for different kinds of rocks compiled by [39]; [43].

Tab. 2.6. Viscoplastic parameters, A and n, for different kinds of rocks after [39] (*); [43](**)].

Material	A [a]	n
Rock salt (*)	1.22E-18	5.3
Granite (*)	3.47E-28	3.2
Granite wet (*)	3.86E-23	1.9
Quartzite (*)	1.29E-24	2.4
Quartzite wet (*)	6.18E-23	2.3
Quartz Diorite (*)	2.51E-22	2.4
Granite (**)	-	3.3
Calcareous rock (**)	-	1.7
Slate (schists) (**)	-	1.8
Shale (clay schists) (**)	-	2.7

a.The parameter A is calculated according to Arrhenius's law (Equation 2.21, see Chapter 2), knowing the activation energy, and for 100 m deep tunnels, considering a gradient of temperature of 1°C each 34 m and an external temperature of 15°C.

The creep law is then derived by integrating Equation 2.20, keeping q constant:

$$\varepsilon_{vp} = a \cdot (q - \sigma_s)^{\beta} \cdot t^{\alpha} \tag{2.22}$$

where

- $\alpha = \frac{1}{1-m}$, [81] quotes the most common values between 0.3 and 1 (note that $\alpha = 1$ is the Norton's law describing secondary creep),
- $a = \left(\frac{A}{\alpha}\right)^{\alpha}$,
- $\beta = \frac{n}{1-m}$.

Lemaitre's law becomes the Norton's law for describing the secondary creep if the time power α is set equal to 1. While, as reported by [83], tertiary creep is usually described by introducing damage effects and mechanical properties decrease to secondary creep laws.

SQUEEZING. Terzaghi describes for the first time squeezing rocks in 1946 [11]. Then, the ISRM, International Society of Rock Mechanics [13], defines the squeezing behaviour of a rock mass as a "large time-dependent deformation, without volume increase, that can occur during tunnel construction and last in time". In some cases it can be quite difficult to recognise if a rock mass has a potential squeezing behaviour. [138] identifies main factors that may contribute to squeezing:

- lithology,
- rock mass strength and discontinuities,
- rock structure orientation,
- initial stress state ($\sigma_0 = \gamma_{rm} \cdot H$, with H = overburden),
- construction method and procedures,
- tunnel supports.

Squeezing case histories are described in [138]; [12]; [13]. Usually, squeezing rock masses are schistose and foliated rocks as faulted shales, marls and argillaceous phyllitic schists, and materials with a considerable amount of clay. When a tunnel is excavated in a strongly fractured or crushed rock mass it can be affected by squeezing even with low overburden. Moreover, the tunnel convergence increases if foliation or discontinuities are parallel to the tunnel axis. Anyway, the major factor that contributes to squeezing is the competency factor (i.e the ratio of in-situ stress σ_0, and rock mass strength σ_{cm}). [69] proposed a method for evaluating the strain ε, knowing σ_0, σ_{cm} and the internal support pressure p_i, and estimating tunnel squeezing potential:

$$\varepsilon = 100 \cdot \left(0.002 - \left(0.0025 \cdot \frac{p_i}{\sigma_0}\right)\right) \cdot \frac{\sigma_{cm}}{\sigma_0}^{\left(2.4 \cdot \frac{p_i}{\sigma_0} - 2\right)} \qquad (2.23)$$

When the evaluated strain ε is bigger than 2.5% the tunnel may show severe squeezing problem. Thus, according to them, usually, squeezing problems may be observed in deep tunnels (i.e. high values of σ_0) excavated in weak and heterogeneous rock masses (i.e. low values of σ_{cm}).

Usually, squeezing may occur when the rock mass around the excavation is yielded. [11] describe three possible tunnel failures due to squeezing rock masses:

1. complete shear failure, observed in ductile rock masses;
2. buckling failure, observed in metamorphic rocks (e.g. phyllites and mica-schists) and in thinly bedded ductile sedimentary rocks as mudstone, shale, sandstone, siltstone and evaporitic rocks;
3. shearing and sliding failure, normally observed in thickly bedded sedimentary rocks.

Moreover, they proposed an analytical model for estimating stress and strain fields around circular tunnels in squeezing rocks. This solution is based on the analogy between the axial stress-strain response of a tri-axial compressive test and the tangential stress-strain response of rocks surrounding tunnels.

SWELLING. As it is possible to see by comparing degradation potential values reported in Table 2.5, when clay and anhydrite formations interact with water they may show exceptional changes due to both physical and chemical processes. The ITA-AITES glossary [source: http://www.ita-aites.org] describes a swelling material as “Material that expands in volume by absorbing or adsorbing water so that it tends to move into a tunnel opening or to exert great pressure upon the supports.” Thus, while squeezing may happen in every kind of rock masses, swelling needs two main conditions to occur: potentially swelling minerals and water presence. Moreover, as affirmed by [8]; [50], swelling and squeezing processes may affect simultaneously only weak rock masses. It is possible to identify three swelling processes:

1. Clay water absorption and consequent volume increase (e.g. Belchen tunnel in Switzerland): it is the most frequent and faster process. In his PhD thesis [17], distinguishes between inter-particles swelling which can affect any kind of clay, with a very small volume increase, and interfoliar swelling, which may result in quite important volume increase and characterises marls and clays with swelling minerals (e.g. Smectite, Montmorillonite).
2. Anhydrite hydration (e.g. Bozberg Tunnel in Switzerland; also in [63]): together with gypsum and volume increase, this reaction develops aggressive agents that may corrode concrete lining as described in paragraph 2.2.2.
3. Iron sulphide (pyrite) decomposition with volume increase.

Rock mass swelling may cause important disorders during construction as well as after several years of operation. As noted by [8]; [30], the main symptom is pavement and walls foundation deformation due to invert heave up (see Figure 2.11). This effect is probably due to the fact that in a tunnel section the bottom (invert) is not subjected to any load from above.

Fig. 2.11. Invert heave and side wall foundation deformation (courtesy of État de Vaud).

As reported by [23], swelling degree mainly depend on:

- thickness of the swelling ground under tunnel foundations,
- water circulation around the tunnel,
- construction method.

As a matter of fact, before tunnel construction, the swelling potential of the rock mass is completely undeveloped. Then, when there is a water source, the excavation and the consequent changes of stress field around the tunnel may activate the process. As described in [8], intact swelling rock masses usually have low values of permeability and a certain damage due to tunnel excavation is, thus, necessary for water circulation around the tunnel and to activate the swelling process, especially beneath the tunnel floor. Possible solutions for reducing rock mass swelling behaviour and related tunnel disorders are:

1. avoid water drilling during construction
2. concreting the invert as soon as possible
3. use circular excavation sections or put compressible material under the tunnel floor.

A detailed review about rock masses swelling behaviour modelling is given in [8]; [23]. For example, as reported in [23], in the framework of the convergence-confinement method, Gysel (1987) modelled swelling as a reversible problem (i.e. elastic rock mass), by using a volumetric strain increase $\Delta\varepsilon_v$:

$$\Delta\varepsilon_v = k_g \cdot \left[1 - \frac{\left(\log\left(\frac{1-\nu}{1+\nu} \cdot 3 \cdot p\right)\right)}{\left(\log\left(\frac{1-\nu}{1+\nu} \cdot 3 \cdot p_g\right)\right)}\right] \tag{2.24}$$

where, in plane strain conditions, the volumetric strain is:

$$\varepsilon_v = \varepsilon_r + \varepsilon_\theta \tag{2.25}$$

while the average stress p is:

$$p = \frac{\sigma_r + \sigma_\theta + \sigma_z}{3} \tag{2.26}$$

and the maximum swelling pressure p_g:

$$p_g = \frac{\sigma_g}{3} \cdot \frac{1-\nu}{1+\nu} \tag{2.27}$$

where σ_g is the swelling pressure. k_g represents the free swelling and it is a function of C_g (Equation 2.28), the swelling index introduced by Huder-Amberg in 1970:

$$k_g = -(C_g \cdot (\log\sigma_{v0})) \tag{2.28}$$

where σ_{v0} is the initial vertical stress in an oedometrical test.

According to [8], a correct interpretation of the swelling behaviour should take into account hydro-mechanical coupling. Indeed, swelling potential is caused by changing natural stress conditions around the excavation. As a matter of fact, when a tunnel is excavated compressive zones arise at tunnel side walls, and traction zone at tunnel crown and invert. Moreover, an hydraulic gradient may generate and water starts moving from compressed zone to relaxed ones and may cause invert heave up. For describing this phenomenon he proposed a complete coupled hydro-mechanical model with numerical solution. In this model, two swelling parameters are introduced and the rock mass is modelled as an elastic-perfectly plastic material with Mohr-Coulomb yield criterion.

Other actions

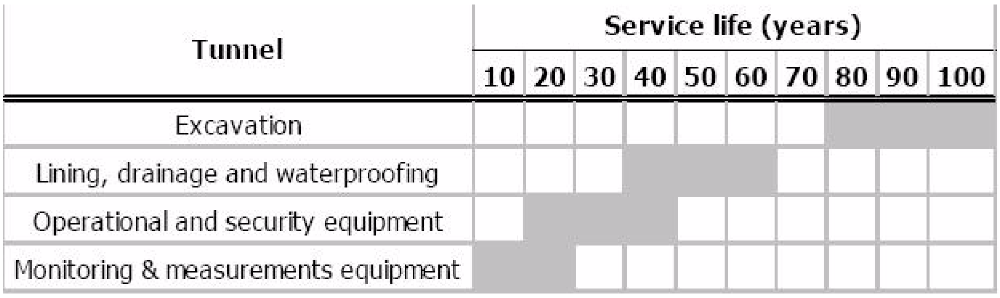
CONSOLIDATION. The classical soil mechanics expressed by Terzaghi's criterion (see Equation 2.29) differentiates between the contribution of solid skeleton and pore fluid to the total stress σ which remains constant with time. Under the hypothesis of non-compressible phases, the consolidation, is the consequence of interstitial pressure u dissipation and σ' effective stress increase.

$$\sigma' = \sigma - u \tag{2.29}$$

Thus, as most of deep tunnels are excavated in water saturated rock masses the re-equilibrium of pore water pressure may change the long term load that the lining has to bear. [58], in his PhD thesis, observed that both the hydro-mechanical coupling during excavation and the pore water pressure re-distribution after construction have a significant effect on long term pressure and displacement. Moreover, he developed a new design method in the framework of the convergence-confinement method for tunnels excavated in low permeable saturated rock masses. Considering both permeable and impermeable linings, he showed that, as the minimum pressure in the long term curve with an impermeable lining is represented by the pore water pressure, the water boundary conditions (i.e. waterproofing system) strongly influence the long term equilibrium of a tunnel.

2.3 Service life and durability

As any structure, a road tunnel must assure users safety and serviceability during all its service life. But, compared to other structures, underground constructions have a longer service life. As a matter of fact, as reported for example in Swiss Norm [135] about tunnel design and construction (Figure 2.12), each tunnel part has a specific service life duration and values change considerably from the excavated rock mass to the electromechanical equipment.



Tunnel	Service life (years)									
	10	20	30	40	50	60	70	80	90	100
Excavation								▨	▨	▨
Lining, drainage and waterproofing				▨	▨	▨				
Operational and security equipment		▨	▨	▨						
Monitoring & measurements equipment	▨	▨								

Fig. 2.12. Ideal service life for each part of a road tunnel according to [135].

During tunnel service life, several factors (e.g. age, material quality, weathering actions) may contribute to reduce structure durability. Moreover, load conditions, rock mass behaviour and environmental actions may change significantly with time. During operation, disorders (e.g. cracks, deformations and water damages) may reveal time dependent pathologies. When a tunnel begins showing disorders it is necessary not only to investigate in order to identify the pathology origins but also, at least in some cases, to operate an upgrade of the structure by means of conservation procedures. In particular, as it is described more in detail in Chapter 3, structural and general aspects should be checked by means of regular inspections. Underground structures survey allows the tunnel owner to follow the time evolution of the structure. Thus, starting from specific initial conditions, evaluated by means of construction data, tunnel durability should be estimated by taking into consideration both degradation processes, which mainly depend on environmental and operational conditions, and conservation activities. This kind of approach can be used for each part of the tunnel and allows to compare expected service life (i.e. from effective norms) to tunnel durability.

2.4 Conclusions

By definition, deterioration is a performance decrease. According to [1], ageing, weathering and structural damages due to external and internal causes may affect the tunnel performance on the long term. Different degradation processes develop depending on the location (e.g. intrados vs. extrados; crown, side walls,...) and on the material used. For example, in contrast to visible surface phenomena mainly due to operational conditions, though less accessible for investigation, the interface between the tunnel lining and the excavated rock mass is susceptible to deterioration due to ground water action.

By taking into consideration Swiss National Roads tunnel features, this chapter provides a detailed literature review about main tunnel degradation processes. As summarised in Table 2.2 and Table 2.3, tunnel long term behaviour depends on both ageing and weathering processes together with other physical and mechanical actions. Lining disorders, identified during tunnel inspections, may be caused by environmental factors (e.g. moisture, temperature) and operation conditions (e.g. traffic, de-icing salts), as well as by rock mass degradation. Several pathologies may appear in the form of the same symptom affecting visible parts of the tunnel (i.e. lining intrados, drainage system, gutter, ventilation slab, walls foundations and track).

A “wide” range of chemical reactions depending on lining material quality, ground water chemical composition and operational conditions (i.e. tunnel atmosphere) has been presented. Furthermore, changes in external loads due to both rock mass delayed behaviour and hydrological conditions modification have been illustrated. Disorders are described as symptoms of typical road tunnels pathologies. This procedure allows identifying the main degradation causes. Moreover, when possible, the pathology has been associated with a mechanical model as summarised in Table 2.7 for estimating degradation consequences in terms of long term stability conditions.

Tab. 2.7. Summary of existing models for describing road tunnel pathologies.

Pathology	Effects	Model	Reference
Concrete creep	strain increase	Equation 2.2, Equation 2.3	[28]
Calcium leaching	mechanical properties decrease	Equation 2.8, Equation 2.9, Equation 2.10	[25]; [96]
Sulphate corrosion	mechanical properties decrease	-	[51], [52]; [150]; [124]; [30]; [119]
Chlorides corrosion (reinforced concrete)	mechanical properties decrease	-	[22]; [129]; [33]
De-icing salts attack (plain concrete)	thickness decrease	(Equation 2.7)[a]	[74]
Rock mass weathering	mechanical properties decrease & increase of loading on the tunnel crown	Equation 2.14 & Equation 2.15, Equation 2.16, Equation 2.17	[110] & [24]; [111]
Rock mass consolidation	strain increases	New Design Method[b]	[58]
Rock mass primary creep	strain increases	Equation 2.22	[17]
Squeezing	strain increases	Equation 2.23	[69]
Swelling	strain increases	Equation 2.24, Equation 2.25, Equation 2.26, Equation 2.27, Equation 2.28	[23] (after Gysel, (1987))

a.Though not proposed by [74], this equation describe the evolution of the degraded thickness with time.
b.An example of application of this method is proposed in Chapter 6.

It has been demonstrated that tunnel durability usually concerns complex interactions between rock mass, ground water and concrete lining, as well as traffic, operational and environmental conditions inside the tunnel, together with conservation procedures during service life. As a matter of fact, the long term behaviour of tunnels is strongly dependent on the interaction between excavated rock/ground mass and lining since tunnel construction and during its service life. Thus, in order to evaluate tunnel global conditions (i.e. safety, serviceability and durability), a detailed analysis of both the lining structure and the surrounding rock mass is required. Moreover, for preserving tunnel structural integrity and guaranteeing safety and serviceability during operation, it is necessary to operate regular surveys and conservation practices, as described in Chapter 3.

3. Tunnels Management and Conservation Procedures

A tunnel like any other kind of structures evolves with time. After several years of operation existing tunnels are affected by pathologies caused by ageing, weathering or, in some special cases, by defects due to improper construction techniques. Disorders can be identified by means of both visual inspections and non destructive tests as it is explained in this chapter. According to [30], one of the major problems with tunnels is that the the interface between excavated rock mass and lining (i.e. lining extrados), though it can be considered active parts in the evolutive deterioration process, is not directly visible. This means that in some cases the tunnel real condition can be hidden behind a good quality of the visible surface of its lining. For this reason, make a good diagnostic of the tunnel “health condition” requires to take into consideration several factors:

1. the lining typology (material quality, construction techniques)
2. the excavated rock mass (geomechanical characteristics and excavation techniques)
3. the operational conditions (traffic, temperature...)
4. the presence of ground water and its interaction with the structure (e.g. chemical composition, waterproofing and drainage systems...).

Tunnel degradation may decrease both serviceability and safety levels until it would be necessary to operate with maintenance and repair practices. Moreover, traffic increase and evolution in safety equipment may oblige the owner to adapt the tunnel to new operational requirements. Both diagnostic (e.g. controls and inspections) and maintenance interventions are part of the tunnel conservation which is described in detail in this chapter.

3.1 Documentation on tunnel conservation practices

Manuals and guidelines on tunnels surveys, controls and conservation techniques can be considered important means for identifying and understanding some typical long term pathologies and preserving the tunnel serviceability conditions. Table 3.1 and Table 3.2 show the available literature on structures management and conservation respectively for European and Extra-European Countries.

Tab. 3.1. Available bibliography about structure conservation in different European Countries. In order to have an easy identification in the following, all the sources have been coded with a simple notification: the code of the country followed by a number.

Country	**Document Code**	**Source Documentation**	**Document Type**	**Year**	**Subject**		
					Rail Tunnel	**Road Tunnel**	**General**
Switzerland	CH1	"Maintenance des ouvrages d'art. Méthodologie de surveillance" - Mandat de recherche OFROU - R. Favre, D. Andrey, R. Suter	PhDThesis/handbook	1987			X
	CH2	"Le diagnostic des ouvrages de génie civil" - Office fédéral des questions conjoncturelles PI-BAT	guideline/analyse	1993			X
	CH3	"Maintenance des ouvrages en tunnel" - Office fédéral des questions conjoncturelles PI-BAT	guideline/analyse	1994		X	
	CH4	SIA 469 - "Conservation des ouvrages"; SIA 262 - "Construction en béton"; SIA 260 - "Bases pour l'élaboration des projets de structures porteuses"	norm	1997/ 2003			X
	CH5	"Directive: Surveillance et entretien des ouvrages d'art des routes nationales" - OFROU	guideline	1998/ 2005		X	X
	CH6	"Directive: Prise en considération de l'entretien dans l'élaboration des projets et lors de la construction des routes nationales" - OFROU	guideline	2002		X	X
	CH7	"Conservation des tunnels CFF SA" - CFF	guideline	2004	X		
France	FR1	"Instruction technique pour la surveillance et l'entretien des ouvrages d'art" - Direction des routes et de la circulation routière	guideline	1980		X	X
	FR2	"Ouvrages souterrains conception, réalisation, entretien - Chapitre 11: Entretien et réparation des tunnels" - Presses de l'École Nationale des Ponts et Chaussées	book	1988	X	X	
	FR3	"Guide pour la surveillance, l'entretien, la conservation des tunnels routiers" - CETU	handbook	1998		X	
	FR4	"Diagnosis methods for lined tunnels" - WG14 AFTES	recommendation	1984/ 1999	X	X	
	FR5	"Guide de l'inspection du génie civil des tunnels routiers. Du désordre vers le diagnostic" - CETU	handbook	2004		X	
Italy	I1	"Manutenzione e riparazione delle strutture in sotterraneo, metodi di indagini non distruttive, tecnologie e materiali speciali" - Atti della giornata di studio - Milano 30/04/1993	conference proceedings	1993	X	X	
	I2	"Adeguamento, manutenzione e arredo delle opere in sotterraneo esistenti" - Atti del convegno Infravia - Verona 11/05/2000	conference proceedings	2000	X	X	
	I3	"Upgrade of ANAS tunnel" - S.Orsini	conference paper	2001		X	
	I4	"Rehabilitation of Highway Tunnels:Techniques and Procedures" - L.Locatelli, G.Di Marco, C.Zanichelli, P.Jarre	conference paper	2001		X	
	I5	"The ANAS Geographic Information System Tunnel Characterization" - S.La Monica	conference paper	2001		X	
Portugal	P1	"Development of methodologies to support the safety control of old railway tunnels" - C.Silva, L.R.Sousa, E.Portela	conference paper	2001	X		
UK	UK1	"Maintenance of Road Tunnels" - The Highways Ag., Scottish Ex.Develop.Dep., Welsh Ass.Gov.LCC, The Dep.for Reg.Develop.Northern Ireland	guideline	2003		X	

Tab. 3.2. Available bibliography about structure conservation in Extra-European Countries. In order to have an easy identification in the following, all the sources have been coded with a simple notification: the code of the country followed by a number.

Country	Document Code	Source Documentation	Document Type	Year	Subject		
					Rail Tunnel	Road Tunnel	General
USA	US1	"Tunnel inspection Manual" - Federal Transit Administration	guideline	2003	X	X	
	US2	"Tunnel Maintenance and Rehabilitation Manual" - Federal Transit Administration	guideline	2003	X	X	
	US3	"Tunnel Rehabilitation in North America" - H.W.Parker, R.A.Robinson, P.M.Godlewsky, W.A.Hultman, R.J.Guardia	conference paper	2001		X	
Japan	J1	"A case study on the period for the economically optimal repairs of RC tunnels suffering of chloride damage" - T.Yamazaki, Y.Tsuburaya	conference paper	2001		X	
	J2	"Tunnel maintenance in Japan" - T.Asakura, Y.Kojima	article	2003	X		
South Africa	SA1	"Inspection, maintenance and repair of tunnels: international lessons and practice" - J.A.Richards	article	1999	X	X	

As this work mainly focuses on road tunnels the bibliographical research has been done principally in that field. Comparing all sources it is possible to made some considerations:

1. There's a general lack of specific norms for tunnels, the majority of norms and guidelines proposes procedures valid for structures in general.
2. Tunnel maintenance handbooks, especially for what concerns road structures, are relatively recent. Due to the age of road network (see Chapter 1), in fact, the most remarkable differences between road and railway tunnels are related to the age of development and construction of the two networks. Consequently the construction techniques and building materials quality and type are relatively different, so are the pathologies. In general, the effects of delayed behaviour often require a long period to become visible: as a matter of facts, tunnels are characterized by having hidden the interface between the rock mass and the lining, which can be considered as the most critical and active part of the whole structure in terms of exposition to long term deterioration (decay).
3. In Switzerland, France, United Kingdom and, recently, USA important sources are guidelines and handbooks published by the Transport Department. In particular, in Switzerland, the directives are written by OFROU in charge of road infrastructure management and CFF for what concerns railway tunnels.
4. In France, as previously seen in Table 1.1 on page 21, tunnels are characterised by having a particularly remarkable age (i.e. 20% of road tunnels are older than 100 years). The majority of bibliographical sources (guidelines) dealing with road tunnels, are written by CETu (Centre d'Etude des Tunnels). It is a governmental institute in charge of surveying and controlling National Roads Tunnels since 1974. Other general documents about maintenance and rehabilitation of underground structures come from the ENPC (Ecole Nationale des Ponts et des Chaussées). Concerning railways SNCF has, since 1973, specific recommendations for ruling survey and rehabilitation of tunnels.
5. In Italy, for what concerns highways, each owner can apply his own conservation procedure, thus it's quite difficult to find general handbooks and guidelines. Anyhow, in Italy, as in Portugal since 1990, a fairly large amount of papers about methodologies of tunnel inspection and maintenance and about real examples of rehabilitation and repairs has been written.
6. Related to Extra European Countries, articles and conference papers are an interesting source of information about tunnel inspection techniques, maintenance and repairs, for example, in South-Africa and Japan.

3.2 Tunnel management

After design project, construction and acceptance, a tunnel becomes operative. Then the tunnel owner operates in two different but interactive domains (see Figure 3.1):

- Operation,
- Conservation.

During the service life a tunnel must be effective in operation without creating traffic problems to the whole road network. The tunnel, seen as a structure, must fulfil its design/conception purposes without creating restraints to normal users or safety problems. During operation, the tunnel owner should, thus, perform a regular activity of preventive maintenance to avoid deficiencies and pathologies development such that each tunnel can "work" (continue to function) as originally designed.

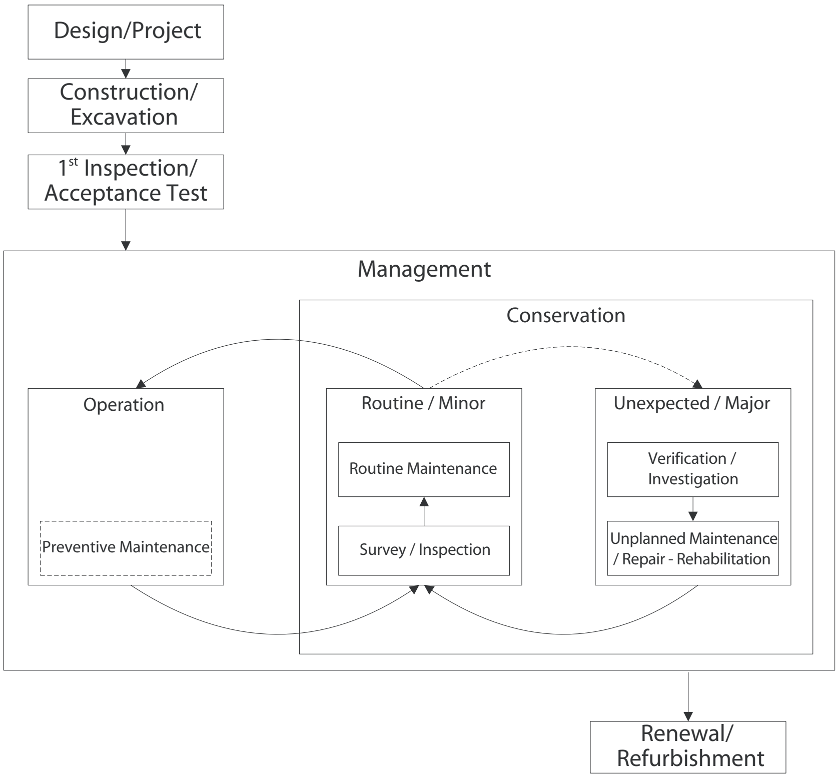


Fig. 3.1. Tunnel life and management (after [54]).

The main objectives of tunnel conservation are:

- Maintaining the tunnel condition above an acceptable serviceable limit, (serviceability)
- Guaranteeing structural and human safety in operation,
- Assuring tunnel durability,
- Guaranteeing the tunnel economical value [100],
- Controlling any environmental interaction.

According to these goals, the tunnel owner should regularly perform a technical survey of the structure and decide about ordinary procedures of maintenance and/or major repairs. As shown in Figure 3.1, conservation consists of two major action fields [147]:

- Routine Conservation, which groups all the technical activities that assure a normal tunnel operation, as survey/inspection and routine maintenance;
- Unexpected Conservation, which groups pathology identification by means of verification and major repairs.

Survey activities are necessary for evaluating tunnel conditions and adopting appropriate conservation procedures [103]. Survey activities comprehend ordinary observations, technical inspections and monitoring.

Routine Maintenance is the technical aspect of Routine Conservation. This kind of maintenance is necessary to avoid ageing problems and to repair some small disorders due to ordinary tunnel operation.

If during survey activities extended damaged areas are identified, repairs might be necessary to prolong the life span of the tunnel. Any kind of structural degradation, which could compromise the safety, results in an interruption or in a limitation of the ordinary operation and in urgent repairs. Otherwise, the ordinary procedure requires structural safety verification and further investigation, in order to discover the problems origins, and a repair design project for making an efficient work.

On the other hand, any kind of intervention that modifies the original structure at the end of its service life is not considered as a conservation activity but as a modification (i.e. Renewal/Refurbishment).

In Switzerland, the [100] guideline proposes a procedure rather similar to the above-explained methodology (Figure 3.1):

1. Survey,
2. Verification (Structural Safety),
3. Diagnosis and Intervention Design Project,
4. Repair/Rehabilitation.

By means of verification, which consists of detailed investigations and structural analyses, it is possible to identify the real extension of the damaged area and to establish the possible causes of deterioration. Once verification is completed, a diagnosis about the possible modes of failure and the tunnel future life expectancy is done. The diagnosis leads the tunnel owner to choose and, then, realise appropriate maintenance, reactive rehabilitation procedures, repairs and, for major interventions like structural improvements, pre-planned refurbishment and renewal.

As it is shown in Figure 3.2, if the tunnel doesn’t fail due to a sudden problem, caused by external factors or by important structural deficiencies at construction, the quality level gradually deteriorates. During service life the improvements due to routine maintenance can’t change the quality level of the whole structure but this kind of intervention is necessary to guarantee the tunnel normal operation by slowing (or prevent in the case of preventive maintenance) the deterioration process. When the tunnel owner, in order to cope with major problems, operates interventions of rehabilitation and repair, the result is an extended service life. On the contrary, if no repair or rehabilitation is done, the tunnel quality level reaches a minimum value (i.e. serviceability limit state). At this moment it is necessary to operate an urgent repair in order to avoid the decrease of the quality level to the ultimate limit state which means the failure of the structure. In that case, both structural and users safety are no longer assured and a complete tunnel refurbishment is required. In other cases, at the end of the tunnel's estimated service life, a complete tunnel renewal is necessary in order to extend operation, for example by adapting the tunnel to a new road network conception or to new traffic needs, preserving safety in operation and service quality.

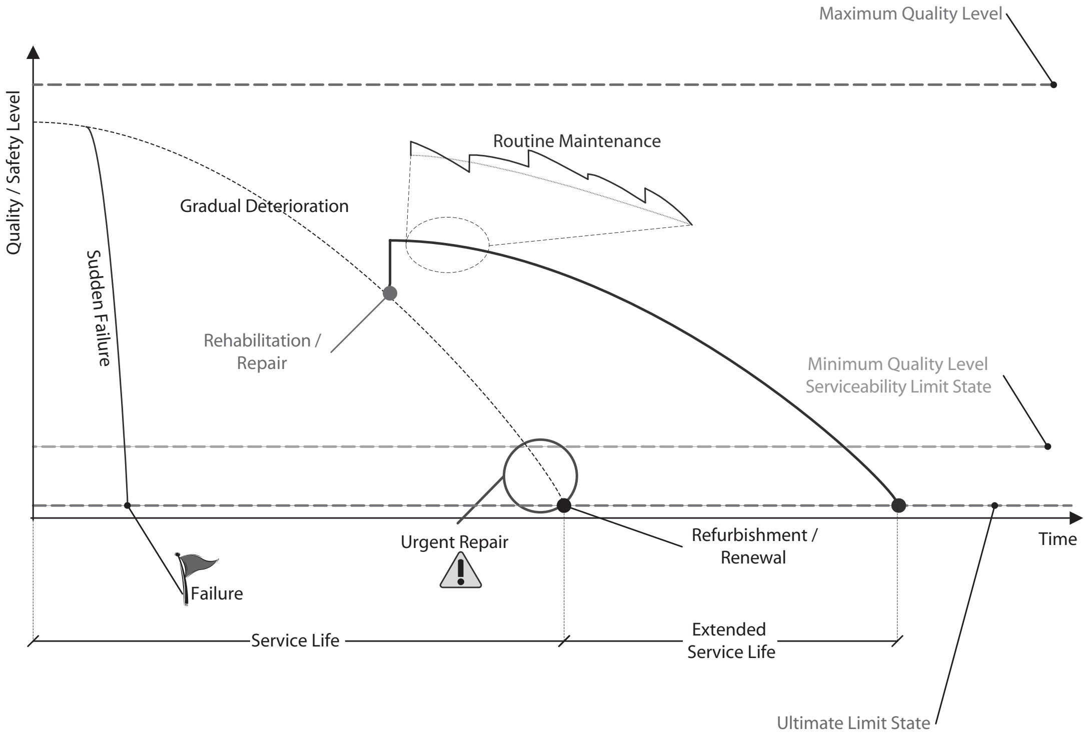


Fig. 3.2. Quality level of the tunnel and service life.

As normally a tunnel is supposed to have a quite long service life (see Chapter 2), tunnel construction is very short compared to service life. Figure 3.3 shows the tunnel life's timeline since excavation till the end of its estimated service life. Any kind of conservation procedure is represented as an event that crosses the tunnel timeline. Only monitoring that can be performed during all tunnel service life, is represented as a parallel line to the operation's one. The survey activity is represented by periodic principal inspections (PI). During the interval between two principal inspections the tunnel is controlled by means of routine inspections (RI) and routine maintenance is performed regularly during operation. In particular conditions, when a certain disorder is identified, it may be necessary to perform local control the structure by means of special inspections; while, when the global tunnel conditions are critical, investigations, laboratory tests and detailed structural analyses are done during verification. If the tunnel has reached an unacceptable safety/serviceability level urgent repairs are required, otherwise if by means of verification typical tunnel pathologies are recognised it can be useful to operate repair and rehabilitation in order to slow the deterioration process (see also Figure 3.2).

In the case of Swiss Road Tunnels, according to [100] principal inspections should be performed once every 5 years, as it is represented in Figure 3.3. Anyway, the same timeline could represent any tunnel in any country by changing inspections frequency as compiled in Table 3.3 for different countries.

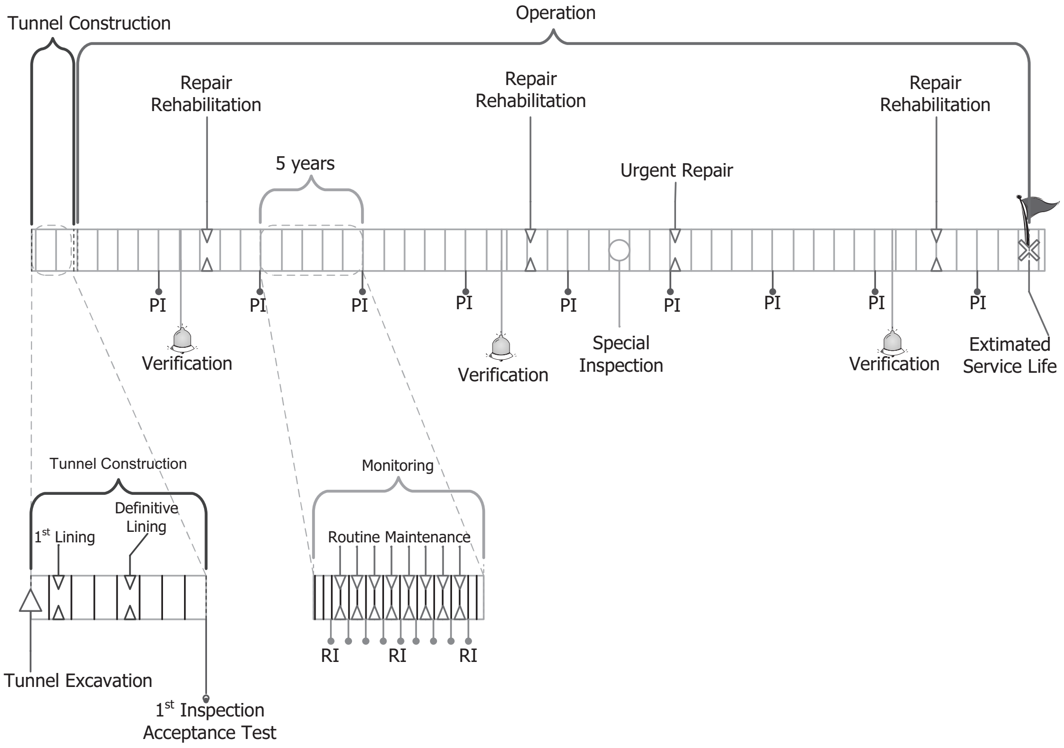


Fig. 3.3. Tunnel life timeline.

Starting from the available documents, the following paragraphs summarise the comparison about inspection and maintenance methodologies for railway and highway tunnels in different countries. The next sections are organised according to the general conservation procedure recommended to the tunnel owner:

1. survey, investigation and eventual monitoring (see Section 3.2.1),
2. tunnel condition diagnosis (see Section 3.2.2),
3. intervention design and operation (see Section 3.2.3, Section 3.2.4 and Section 3.2.5),
4. data collection (see Section 3.2.6).

Some of the analysed documents (Table 3.1, Table 3.2) contribute only qualitatively to the comparative study as they deal more with safety measures and equipment (e.g. Italy: I3 [109] and UK: UK1 [67]) or with specific methods for data collection and treatment (e.g. Italy: I5 [84] and Portugal: P1 [136]).

3.2.1 Survey activities

Survey activities comprehend:

- Observation (regular visual inspection/ ordinary general inspection): it is a sort of operational supervision by means of regular (i.e. daily, weekly, monthly, yearly) visits. It allows recognizing local and surface degradations.
- Technical Inspection (periodic detailed inspection): it is necessary to confirm the tunnel conditions evaluation, recognise the type of deterioration and damage and evaluate the degradation rate.
- Monitoring (measurements control): it is a survey by means of specific measurements; it plays an important role in the management of deteriorating structures. By following the evolution of identified pathologies it's possible to keep under control abnormal behaviours of the structure and prevent danger. The sections to put under control, the measurements types and the instruments are defined according to the data collected during tunnel inspections.

By means of tunnel inspections the tunnel owner aims at:

- Making a systematic and periodic control of the tunnel in order to follow its evolution with time,
- Recognising disorders and identify their origins,
- Determining which parts of the structure are more easily affected by degradation,
- Evaluating changes in tunnel operation,
- Giving important and necessary information for operating tunnel maintenance and rehabilitation,
- Making possible urgent intervention in order to assure the tunnel safety and serviceability during operation.

As mentioned previously, compared with other structures, the effects of deterioration in tunnels are generally observable with some delay.

Table 3.3 shows for several countries the different inspection types according to frequency of occurrence and objectives.

Tab. 3.3. Inspection type and frequency in different countries. The Document Code is the same used previously in Table 3.1 and Table 3.2.

		Inspection Frequency		
		General	**Principal**	**Special/Intermediate**
Country & Document Code	**Network**	**(routine)**	**(planned)**	**(not planned)**
Switzerland (CH1, CH3, CH4, CH5, CH7)	Road	max. once a year	once every 5 years	if particular problems
	Rail	once a year and continuous monitoring	once every 5 years (max. once every 10 years)	if particular problems
France (FR1, FR3, FR5)	Road/Rail	once a year and continuous monitoring	once every 6 years (min. 3 years, max. 9 years) once every 3 years inspection for evaluation IQOA (Section 3.2.2)	if particular problems
Italy (I2, I4)	Road/Rail	once a year and continuous monitoring	once every 5 years	if particular problems
Portugal (P1)	Rail	continuous monitoring	-	-
UK (UK1)	Road	continuous monitoring; safety inspection: monthly; surface inspection and general inspection: once every 2 years; equipment inspection: the frequency, in this case, depends on the equipment	once every 3 years not exceeding 6 years (exceptional case = 5/10 years)	if particular problems
USA (US1, US2)	Road/Rail	weekly or monthly walk-through general inspection (the period is a function of the equipment type)	once every 5 years (2 years for older tunnels)	if particular problems
Japan (J1, J2)	Road	once a year and continuous monitoring at every 5 m of the tunnel's length	-	if particular problems
	Rail	once every two years	once every 10 or 20 years (it depends on importance of the network)	if particular problems
South Africa (SA1)	Road/Rail	-	the frequency depends on type of facility and operating environment	if particular problems

By analysing Table 3.3, it is possible to observe that in the majority of countries, there are three kinds of inspections (the Document Code is the same used previously in Table 3.1 & Table 3.2):

1. General (Routine) Inspection (RI in Figure 3.3): it is a routine visual survey. It focuses on visible and structurally important parts of the tunnel searching for evident evolution marks. It takes place with variable frequency, normally yearly (see Switzerland: CH1, CH3, CH7; France: FR1, FR3, FR5; Italy: I2 and Japan: J2) but, in some countries (UK: UK1 and USA: US1, US2) it is performed weekly or monthly.

2. Principal Inspection (PI in Figure 3.3): it is a detailed survey of all the accessible parts of the tunnel. It is compulsory in order to guarantee the serviceability of the tunnel. This kind of inspection has a variable schedule between 3 years (see in particular UK: UK1 and France: FR3, FR5) and 20 years (as in Japan: J1) depending on the problems identified in the tunnel and on their evolution rate. Special cases of very old tunnels can require a Principal Inspection once every two years (see USA: US1).
3. Special/Intermediate Inspection: it is rather a local control, limited to the zones where a certain disorder is identified. Normally, it has a variable schedule depending on the disorder type, severity and location and it is performed between two Principal Inspections in order to follow the evolution of the identified disorders. In some ways, this kind of survey can be considered as the first step of a verification in the repair procedure. Moreover, when a certain problem is identified, Special Inspections may be required.

CONTROL TYPES & TECHNIQUES. Inspections are an important information source in order to point out which parameters play an important role and influence tunnel stability conditions and long term behaviour. As it is shown in Table 3.4, several documents describe different types of control usually performed and the techniques used during tunnel inspections. The control type can change in function of the type of parameter that has to be checked for recognising pathologies. Most manuals and guidelines report the following list of techniques:

- Visual Inspection: it is the most common and simple general control to identify superficial defects and, in some cases, recognise potential danger. This kind of survey is limited to the visible surface of the lining (intrados). It is quite useful to identify defects such as collision damage, humidity, ice and/or stalactite formation, spalling, cracking, joint sealant failure, efflorescence, leaching, weathering, corrosion and deformation (as described in Chapter 2). Visual inspection techniques are crack presence control and width evolution survey, lining deformation control, hammer testing.
- Geometrical Inspection: it is a shape/profile control used to detect eventual displacement of the lining intrados and abnormal behaviour of the tunnel. It is performed by means of special instruments. The tunnel geometry can be controlled by profile and topographical measurements
- Convergence and Stress Monitoring: it is recommended in case of particular geological conditions for some representative cross sections. It requires specific instruments and allows to follow the evolution of the tunnel closure with time. Moreover, as it happens already during the construction process, tunnel convergences can be used as a tunnel instability warning.
- Water Inflow Measurement: as water can be considered one of the principal causes of tunnel degradation, it is quite important to control quantity and quality of water leakage through the tunnel lining. Moreover, in order to avoid unexpected loads acting on the lining it is important to inspect regularly the condition of the drainage system. The presence of water is controlled by means of water inflow measurements. Laboratory tests, together with drainage inspections can be performed in order to measure water composition and quality.
- Material Investigation: during verifications the lining can be investigated by means of geophysical measurements and non-destructive methods as Ground Probing Radar (GPR), Optical Fibres, Impact Echo and Thermography. These methods are quite often associated to material investigations and laboratory tests on core samples.

Each country specifies that controls of electro-mechanical equipment should be regularly performed. As a matter of facts, tunnel equipment, compared to the structure itself, have a shorter service life (e.g. Figure 2.12, Chapter 2), and periodic performance controls are required to assure safety in operation. Though electro-mechanical equipment is not object of the present study, it is possible to affirm that its frequency mainly depends on operation conditions and safety requirements.

Tab. 3.4. Control types in different countries. The Document Code is the same used previously in Table 3.1 & Table 3.2.

Country & Document Code	**Network**	**Visual**	**Geometrical**	**Convergence and Stress Monitoring**	**Water Inflow**	**Material Investigation**
Switzerland (CH1, CH2, CH3, CH4, CH5, CH7)	Road	X	X	X	X	X
	Rail	X	X	X	X	X
France (FR1, FR2, FR3, FR4, FR5)	Road	X	X	X	X	X
	Rail	X	X	X	X	X
Italy (I2, I3, I4)	Road	X	X	-	-	X
	Rail	X	X	X	X	X
Portugal (P1)	Rail	X	X	-	X	-
UK (UK1)	Road	X	X	-	X	X
USA (US1, US2, US3)	Road	X	X	-	X	X
	Rail	X	X	-	X	X
Japan (J1, J2)	Road	X	X	-	X	-
	Rail	X	X	X	-	X
South Africa (SA1)	Road/ Rail	X	X	X	X	X

The comparative analysis of documents about conservation procedures shows how most control types are common to several countries. As mentioned above, during inspections, different control types can be performed using specific techniques (Table 3.5). The results can be outlined as follows:

1. By general inspection the tunnel owner wants to evaluate the general conditions of the tunnels by means of simple controls, normally a visual survey of the tunnel intrados surface. During general inspection it is possible to identify weathered zone to be further investigated. Visual inspection may reveal changes either in the lining thickness or in the external loads.
2. During principal inspection, it is necessary to point out the major problems that affect the tunnel. It is possible to recognise general instability and movements of the slope where the tunnel is excavated. Moreover, convergence monitoring and load and stress measurements can help in evaluating the stability conditions of the structure.
3. When particular problems are identified it is necessary to evaluate more in detail the tunnel conditions.

Tab. 3.5. Control type and techniques performed during inspections.

Inspection			**Control**	
General	**Principal**	**Verification**	**Type**	**Technique**
X			Visual	cracks evolution control deformation control surface weathering (hammer testing) (temperature condition)
	X		Geometrical	profilometer / intrados geometrical control geometrical and topographical measurements displacements measurements
	X	X	Convergence and Stress Monitoring	rock mass movements stress-strain measurements
	X	X	Water Inflow	water inflow measurement humidity presence of ice chemical composition drainage controls
		X	Material Investigation	geophysics and non destructive methods (GPR, Thermography, Impact Echo; optical fibres) probe holes vault and walls strength core boring and laboratory test

3.2.2 Tunnel condition evaluation

A good decay rate assessment and long term stability prediction are necessary for planning maintenance and repairs. Based on survey results the analysed documents propose several methods for evaluating the general tunnel conditions (see Table 3.6). The principles of the classification methods proposed by the different guidelines are quite similar. After a principal inspection, tunnels are divided in categories, based on a global mark that takes into consideration tunnel general conditions. As one of the most common inspection technique is a simple visual survey, an important role in condition evaluation is covered by aesthetic aspect and structural conditions of the tunnel lining. This kind of evaluation could show its limits when the structure is affected by delayed behaviour of the rock mass and also when the chemical dissolution, caused for example by ground water, starts at the extrados of the lining. Another restriction, related to this methodology, is the lack of information on operational conditions: data about traffic, for example, can be quite useful for a better scheduling of eventual special inspection, verification and/or urgent repairs in order to avoid any kind of hazard.

Tab. 3.6. Tunnel conditions evaluation methods after principal inspection for different countries. The Document Code is the same used previously in Table 3.1 & Table 3.2.

Country	Network & Document Code	Evaluation Method / Note	Remarks
Switzerland	Road (CH1, CH2, CH4, CH5)	evaluation matrix for each pathology (KUBA-MS for bridges)	
		five classes for concrete structures conditions (tunnel lining): 1. good conditions 2. acceptable conditions 3. defective conditions 4. poor conditions 5. alarming conditions	the controls have two thresholds: 1. serviceability limit state (SLS), 2. ultimate limit state (ULS). After verification and investigation, more precise evaluation of damaged zones and estimation of evolution rates
	Rail (CH7)	classification based on general conditions of each part of the tunnel lining: 1. without problems 2. light structural and/or operational problems 3. important structural and/or operational problems 4. serious problems and potential danger	each part of the structure receives this kind of note and by a weighted average it's possible to obtain a global evaluation
France	Road (FR1, FR3, FR5)	two different evaluation types: 1. based on tunnel conditions 2. based on damages evolution rate	
		index called IQOA (Image Qualité Ouvrage d'Art = index of the structure quality). Two different notes are given to the tunnel: 1. for the structure, 2. for the presence of water, both ranging from class 1 when the tunnel is in good conditions to class 3U when there's a need of urgent repairs and with a particular code S when there are safety problems	proposed in 1994 by the CETu (Institute for Roads Control)
Italy	Road/Rail (I1, I2)	rating from 1 (=best condition) to 5 (=worst condition)	it allows to identify the priority of structural refurbishment
	Road (I4)	three incidence indexes of the defect: 1. low 2. medium 3. high	tunnels are periodically checked using an approach based on the assessment of structural and geo-related risks
USA	Road/Rail (US1)	rating from 0 (=worst condition) to 9 (=best condition) for each parts of the structure	once the defect is identified it should be classified as minor, moderate or severe; then, a repair classification is proposed: 1. critical 2. priority 3. routine

Country	Network & Document Code	Evaluation Method / Note	Remarks
Japan	Road (J1)	four classes based on deterioration index	after verification there's a secondary classification based on the size of damaged area: 1. large 2. medium 3. small 4. very small

Disorders and pathologies can be classified for general purposes by type, intensity, damaged area size, incidence, etc. Pathologies can be, thus, hierarchised by a severity criterion based on surface extension, evolution speed and influence on stability condition of the whole structure, this means for the tunnel owner having a simple quantification system of the detected problem. However, the decision about how and when it is necessary to activate repair procedures should take into account also other parameters that influence in an indirect way this first "objective" classification. Within those parameters, for example, there's the structure strategic importance, the traffic conditions and also logistic consequences of maintenance and repair operations (e.g. bidirectional tunnel on an important motorway connection). Thus, when conservation procedures are foreseen it's always necessary to think that an interruption of service on an important communication corridor may mean very high social costs.

3.2.3 Maintenance

Maintenance groups all the simple and routine procedures to increase durability of the tunnel, in terms of length of service life, without changing the tunnel performances. Some of the countries analysed (i.e. Switzerland, Italy and USA), in their maintenance guidelines, distinguish between routine and preventive maintenance (see Figure 3.2 and Table 3.7).

Tab. 3.7. Maintenance in different countries. The Document Code is the same used previously in Table 3.1 & Table 3.2.

Country & Document Code	Network	Maintenance Type	
		Preventive	Routine
Switzerland (CH1, CH3, CH4, CH5)	Road	X	X
France (FR2)	Road/Rail	-	X
Italy (I1, I2)	Road/Rail	X	X
UK (UK1)	Road	-	X
USA (US1, US3)	Road	-	X
	Rail	X	X
Japan (J2)	Rail	X	-

Regular preventive maintenance can be considered as a part of the operation process and has a precise schedule, which changes from country to country and, in some cases, also from tunnel to tunnel, depending on operational and environmental conditions. Typical maintenance schedule is once every 12 months. To ensure serviceability of the tunnel and safe operation of the tunnel, this kind of maintenance includes standard procedures as:

- Tunnel washing;
- Drain flushing;

- Ice removal;
- Tile removal.

Routine maintenance, instead, is a part of the conservation process and includes all the procedures for maintaining the tunnel's performance at a serviceable level. Typical routine maintenance has not a precise schedule. Anyway, this kind of maintenance is associated to simple reactive repairs of faults and disorders identified during ordinary visual inspections but doesn't include major repairs, renewal or reinforcement of structural elements.

The most common procedures are:
- Clean (and rod) through any ground water drainage;
- Remove debris from movement joint seals;
- Repair movement joint seals;
- Clean any debris from walls and gutter;
- Test monitoring and electro-mechanical equipment;
- Remove graffiti.

3.2.4 Rehabilitation/Repair

When inspections and verifications show important damages, ordinary maintenance is not enough to assure serviceability and safety during operation and the tunnel needs structural rehabilitation and repairs. This kind of conservation procedures depends directly on degradation causes. As it is possible to read in technical notes about tunnel rehabilitation (i.e FR3 [29], I4 [88], US3 [117], J1 [151], J2 [9]), repairs performance highly depends on the diagnosis quality and on the possibility of recognising pathologies and degradation factors during inspection activities. Thus, before operating any kind of repair (there’s a precise methodology to follow in order) it is necessary to identify the causes that lead to the recognised pathologies and to make the intervention more effective. In this process it is important to take into consideration all the factors that could influence the behaviour of the tunnel especially on the long term. Any kind of recurrence (cyclic behaviour) should be considered with particular attention and emphasized. Moreover, in order to determine which kind of intervention is the most appropriate, the typical features of each structure have to be taken into account. The major result after repair and rehabilitation is an improvement in tunnel life span.

For example, in the case of protection against water degradation (as described in Chapter 2), it is possible to choose between two types of intervention:
1. prevent/avoid water infiltration, or
2. control water infiltration.

In the first case, the attention will be focused on the waterproof sealing, while, for controlling water inflow, it is necessary to focus on the drainage system. The effects on tunnel lining are completely different: by avoiding water infiltration, in case of total waterproofing system if the drainage system is obstructed, the lining has to bear the overburden and the water pressure, while control water infiltration may increase the wash-out problem. Then, the choice may be done considering tunnel lining conditions, time required for intervention and costs.

An effective repair is, thus, an adaptation to each tunnel of some standard procedures, depending on the identified causes that lead to the disorder, on the deterioration degree and location. Furthermore, the intervention is optimised in terms of goals, costs, delay and interruption of operation.

3.2.5 Refurbishment/Renewal

After a sudden failure or major incident (e.g. fire), or simply at the end of the service life, the tunnel needs important structural improvements. In such conditions, refurbishment is required, in order to increase the tunnel quality/service level. In some cases the renewal of a structure is necessary in order to adapt the tunnel to new operational conditions and requirements. One of the most common interventions during the last years is the renewal of the ventilation system in order to respect new safety requirements during fire events [101]. Concerning tunnels on Swiss National Roads, two representative examples are the Glion tunnel and the San Bernadino Tunnel [149]. Another important renewal operation is the enlargement of the tunnel size in order to satisfy new traffic conditions.

3.2.6 Data collection

Tunnel data should be recorded in an appropriate data base, as Table 3.8 shows for several countries. Indeed, collect and store information about tunnels, after their construction and during operation, may help the tunnel owner to follow the evolution of the structure and identify the possible origins of pathologies. This improves also maintenance and repair planning during tunnel service life.

Tab. 3.8. Methods and tools for recording principal inspection information and inspection data for different countries. The Document Code is the same used previously in Table 3.1 & Table 3.2

Country	Network & Document Code	Data Recording	
		System	Description
Switzerland	Road (CH2)	BDR (Roads Data Base)	General collection of data about roads infrastructures, it collects all kind of information for describing the road network conditions and planning maintenance and repairs
	(CH5, CH6)	KUBA-DB/KUBA-MS	Road structures data base (e.g. bridges, retaining walls, etc.). It contains general data about the structures, their stability conditions, degradation types and materials quality. At present, there isn't a specific data base for tunnels
	Rail (CH7)	Dossier du tunnel	Tunnel data collection, it must be updated after each principal inspection
France	Road (FR3)	DICOS+BDT	Road infrastructure data base with a detailed appendix about tunnels. It contains data about structures since their construction and during operation, as well as inspections data. It is used to manage and schedule tunnels maintenance and repairs
	Rail [Gillan, 2002] (FR1)	RADIS	Data collection from inspections in French railway tunnels (RFF). By means of graphical representations it allows diagnosis of anomalies and estimation of disorders evolution rate
Italy	Road (I5)	National Road File	Tunnel data collection since their construction and during operation (the data are stored in GIS format). The aim is mainly to obtain a tunnel classification related to road security
Portugal	Rail (P1)	MATUF	Computerised system for processing (storing and analysing) available data about railway tunnels
UK	Road (UK1)	Asset Management Software	Computer software (associated with a data base) that assists in fast and effective facility management. It requires general information about the tunnel and up-to-date inspection and maintenance documentation
USA	Road/Rail (US1, US2)	Inventory Data Base	Computerized data base system developed to assist with the storage and processing of tunnel conditions data and to help tunnel owners in maintaining their structures by prioritising repairs
South Africa	Road/Rail (SA1)	Facility Diagnostic File	General information and data collection about tunnels, which is updated after each principal inspection

3.3 Road tunnel conservation in Switzerland

In Switzerland, National Roads are considered as public transportation infrastructures. Their management into the Swiss Confederation is supervised by DETEC, the Federal Department of Environment, Transport, Energy and Communication, and OFROU, the Swiss Federal Roads Authority [100].

The OFROU plans and takes decisions about survey, maintenance, rehabilitation and refurbishment activities and, finally, collects/controls data from inspection and verification reports. At present, tunnel operation is considered as a task of each Cantonal Department for Federal Roads. According to [100], from 2008, the Confederation will cover all the duties as a supervisor and owner of Motorways (RN): thus, in practice, OFROU will be responsible not only of ruling and financing conservation procedures but also of building, operating and technically maintaining the National Roads Network. Figure 3.4 summarises in a flowchart the whole conservation procedure in Switzerland:

- Regular tunnel surveys (e.g. principal inspections once every 5 years and routine inspections once a year as recommended by (12)), are fundamental to follow the evolution of time dependent problems (cracks, deformations and water damages). Specific and periodic controls of structures give information about the tunnel's performance in the future. General and Principal Inspections and, when necessary, verifications are carried out by private inspectors with a particular experience in the field. Inspections and investigations reports are written by the inspectors.
- When no relevant problem is identified the serviceability is guaranteed by means of simple interventions of routine maintenance (e.g. drain flushing, ice and tile removal and tunnel washing).
- If inspections reveal major problems, by means of verification, (i.e. detailed investigations and structural analyses) it is possible to identify the real extension of the damaged area and establish the possible causes of deterioration.
- Once the verification is completed, a diagnosis about the possible modes of failure and the tunnel future life expectancy is done. Tunnel data analysis can be done using several methods and should take into account not only operational and environmental conditions but also typical pathologies identified during survey activities.
- Tunnel conditions, evaluated by means of data analysis from inspection and verification reports, are, then, checked by the supervisor, who, together with the tunnel owner, decides the type and the schedule of intervention (i.e. repair and eventually renewal activities). All information about conservation activities and collected during inspections should be stored and updated in a proper data base (i.e. KUBA-DB for structures in general).
- Technical activities as repair and/or renewal are performed by private companies and supervised by the tunnel owner.

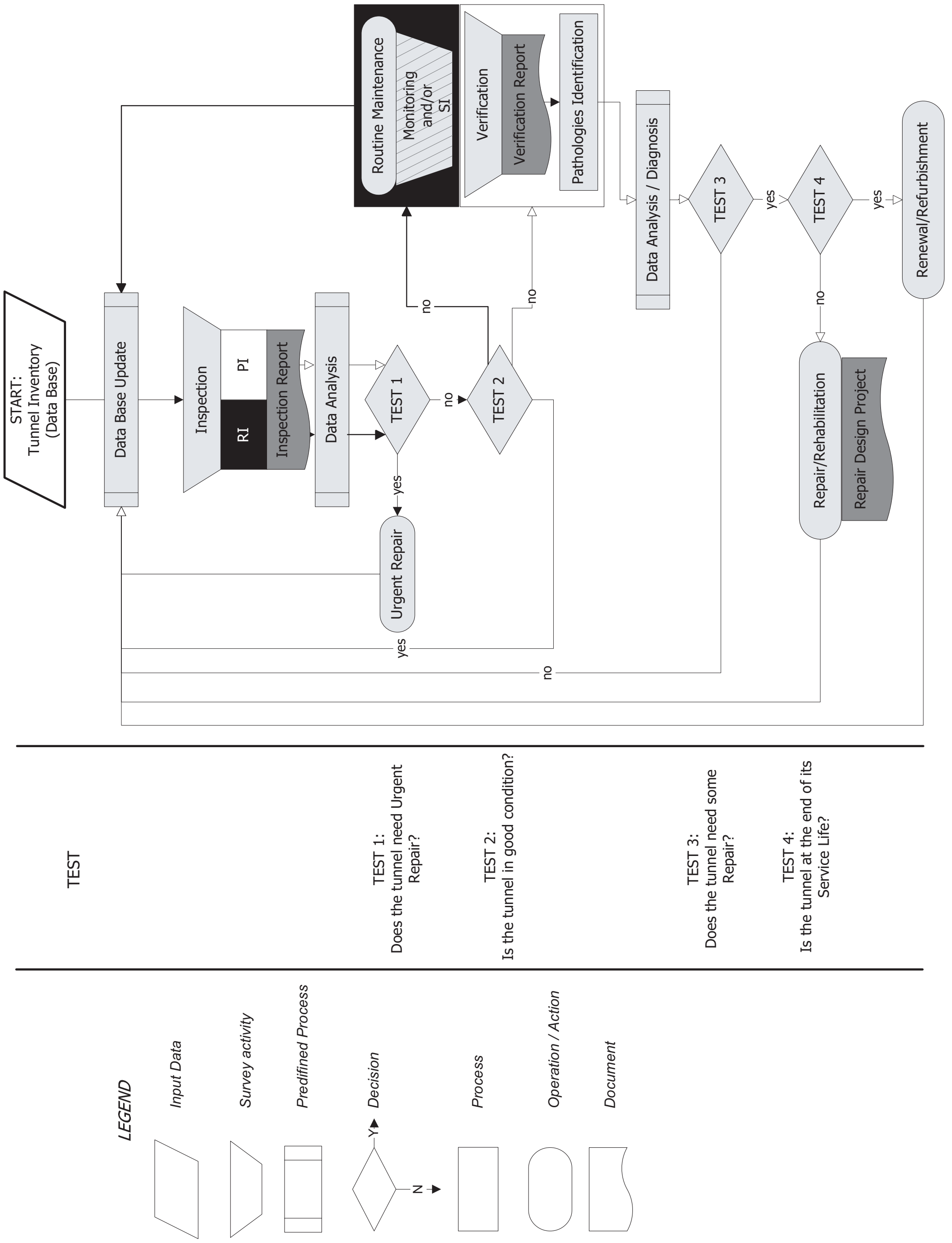


Fig. 3.4. Tunnel Conservation Process in Switzerland. If Survey Activity block is dashed it means that the operation is not compulsory.

3.4 Conclusion

The concept of long term performance (durability) for a tunnel depends on different factors such water action, degradation of the rock mass, of the structural elements and of the electro-mechanical equipment and the consequent interaction with the operation. In order to cope with time effects, it's necessary to apply conservation procedures. This chapter shows how procedures and prescriptions may change from country to country. Particular attention is given to Swiss procedures according the Federal Roads Authority (OFROU). Conservation procedures change according to tunnel timeline. In particular:

- During service life, when deterioration processes (see Chapter 2) modify tunnel serviceability and stability conditions, the tunnel needs maintenance and rehabilitation practices.
- At the end of the tunnel service life, when normal operation and safety conditions are no longer assured, due to an old conception of the whole structure, the tunnel needs refurbishment/renewal. This kind of procedure, for example, becomes necessary when the tunnel needs to be adapted to new operational, safety and/or ventilation requirements [101].

The goals of conservation procedures for tunnels are:

- Maintaining the regular functions of the structure in terms of both serviceability and safety,
- Preserving the tunnel characteristics to guarantee its future life span and its economical value,
- Estimating tunnel's performance in the future.

According to [98] evaluate/predict the long term performance level of a structure is one of the most difficult duties and goals of inspection and investigation activities. In order to achieve conservation objectives, tunnels maintenance and repair schedule still need improvements in accuracy. This comes from the possibility of better assessing the future life expectancy and improving deterioration predictability. Effective repair procedures still need more precise evaluation of actual stability conditions and assessment of remaining strength. Thus, an evaluation of tunnel global condition in terms of serviceability, durability and safety would require a detailed analysis of each part of the lining structure and the excavated rock mass. This analysis should allow a global assessment of the tunnel and an evaluation about its expected life span.

4. Swiss Road Tunnels Data Base (TDB)

Tunnel degradation depends on several factors as shown in Chapter 2. By taking into consideration all tunnel features such as geometry, geological and hydrogeological conditions, age, construction techniques, operational conditions and material quality it may be possible to better evaluate long term serviceability and behaviour. Moreover, as described in the previous chapter (see Chapter 3), effective tunnel conservation mainly depends on diagnosis. A good diagnosis of tunnel conditions is based on a good survey. Thus, collect and store information, since tunnel construction and during operation, may help the tunnel owner in following the structure evolution with time. This chapter describes how a specific data base for National Roads tunnels has been created, pointing out its main features and the problems encountered during data collection. The possibility of integrating data from several sources by means of G.I.S. Tools is also described.

4.1 Tunnel Data Base purposes

As described in Chapter 1, Swiss roads count a fairly large number of tunnels. Considering that the Swiss National Roads began to develop in 1960s, some tunnels have already more than 40 years of operation and may need maintenance to assure everyday serviceability and safety.

As summarised by Table 3.8 in Chapter 3, information about structures on the road network can be collected by means of an appropriate data base. By compiling available literature about road tunnels degradation and conservation procedures, (e.g. [142]; [123]; [70]; [30]), it is possible to outline the main features that characterise an appropriate tool for managing tunnel information:

1. Both past and present (i.e. construction and operation) data are necessary to describe the actual tunnel conditions.
2. Important information about tunnel general conditions comes from survey activities [123]; [55] and [56]; [67].
3. As already described in Chapter 2, according to several authors [70]; [67]; [30], one of the major restrictions for evaluating tunnel conditions depends on the fact that only the the lining intrados can be regularly inspected. In spite of this limitation, a lot of tunnel pathologies develop at the interface between the excavated rock mass and the lining, due to their interaction and to ground water presence. Thus, it is necessary to search for all construction data (e.g. construction method, lining material and quality, drainage and waterproofing systems, etc.) together with geological and hydrogeological information.

The Swiss Federal Roads Authority has a specific Data Base for roads structures called KUBA-DB [104], which, in its actual form, is not convenient for collecting data about tunnels. Thus, in order to have enough information to study the long term behaviour of road tunnels, it has been necessary to create a specific data base for Swiss National Roads tunnels. Three main operations have been necessary for creating this new data base:

1. Analysis of existing data (FGU-SIA Tunnelstatistik, [48]);
2. New concept starting from a detailed literature analysis and advices from CETu (France) and Perss Ingénieurs conseils SA (Fribourg, Switzerland);
3. Data collection by means of an appropriate technical form.

4.2 Analysis of existing data

Since the year 2000, the LMR - EPFL created a data base and collected data for the Swiss Tunnelling Society (FGU, i.e. part of the SIA - Swiss Association of Engineers and Architects), for inventorying tunnels excavated or under construction in Switzerland. This data base counts about 1200 tunnels divided into 4 main categories:

1. Rail tunnels,
2. Road tunnels,
3. Water galleries,
4. Others as supply, access and sewage galleries.

This inventory consists of general information about:

- Tunnel name, location and owner/responsible;
- Technical data: tunnel geometry, excavation method, ventilation facilities;
- Geological data, geological problems and water incomes encountered during excavation.

The National Roads tunnels were first selected among the road tunnels collected in the FGU-SIA inventory, also referring to the official list of underground constructions (i.e. both excavated and cut and cover tunnels) provided by OFROU in 2003. By means of simple statistical analyses, as shown in Figure 4.1, it has been possible to identify some typical features describing National Roads tunnels as for example the age of the structures (upper graph) and their depth (lower graph). The main purpose of this previous classification was to select among the bibliographical sources the most relevant documents for the creation of a suitable data base.

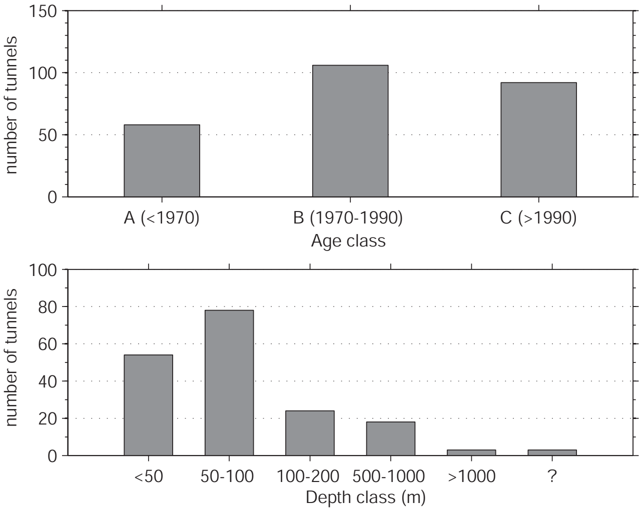


Fig. 4.1. Preliminary statistics about National Road tunnels using information from FGU-SIA inventory. Upper graph: number of tunnels per age class; lower graph: number of tunnels per depth class (the question mark '?' means that the information is not available).

4.3 Technical form and data collection

In order to analyse road tunnels conditions and investigate their main pathologies, it has been necessary to improve and update the information recorded in the FGU-SIA inventory. For this purpose, a specific technical form has been created and sent to each cantonal authority responsible for National Roads tunnels.

A first version of the technical form for data collection was created based on a detailed literature review, with particular attention to documents from CETu [29]; [30], which has a well-known experience in tunnels design and conservation, and sent for a test to État de Vaud. Then, based on tunnel inspectors advices (i.e. S. Nendaz, État de Vaud - Service des Routes Nationales; M.Stempfel, Perss Ingénieurs conseils SA Fribourg, Switzerland; A. Jeanneret, OFROU; people from CETu, France) the form has been improved. Finally, it has been sent to each cantonal responsible for National Roads. Table 4.1 resumes the principal events/steps for creating the technical form

Tab. 4.1. Technical form creation and improvement timeline

Date	Event
07/2004	First concept
11/2004	Presentation to Cantonal responsible of État de Vaud for advice
04/2005	Presentation to OFROU for advice
08-11/2005	First test with 8 tunnels on the A9 (État de Vaud)
11/2005	Improvements by Perss Ingénieurs conseils SA and CETu - Final version
02/2006	Technical form sent to each cantonal responsible for National Roads

The final version of the technical form (Appendix I) is divided into 5 main sections:

1. General information,
2. Construction information and technical data,
3. Geological and hydrogeological information,
4. Operation and environment information,
5. Maintenance information.

The "General information" section is necessary for identifying the tunnel and groups all data about tunnel owner and location as detailed in Table 4.2.

Tab. 4.2. General information.

General Information	Data
IDENTIFICATION:	
	Name
POSITION:	
	Coordinates (X,Y)
	Location (km begin; km end)
	Road
	City
	Canton
RESPONSIBLE:	
	Name / Office
	Address
	E-mail
	Telephone No.

As for other structures, the delayed behaviour of tunnels is affected by several features chosen or encountered at their origin, such as construction method, quality of building materials, geological and hydrogeological conditions. The "Construction information" section (see Table 4.3) groups information about the tunnel initial conditions: structural and geometrical data, technical equipment and other construction details as building materials, construction method. Data about accidents during excavation should be as well provided as they may inform about potential future problems and weaknesses. A "Geological and Hydrogeological information" section (see Table 4.4) completes the information with the geotechnical conditions, including data about the presence and the position of the ground water level. This is compulsory to evaluate the loads acting on the tunnel lining and also to identify the potential tunnel weakest zones from a geotechnical point of view.

Tab. 4.3. Construction information and technical data.

Information	**Data**
CONSTRUCTION:	
Construction Method	Construction year, Excavation techniques
Tunnel Geometry	Shape, Size, Number of tubes, Depth, Length
MATERIALS:	
Rock Mass	Rock Mass Quality during construction
Lining	Support and Lining type and geometry (thickness, length)
Waterproofing and Drainage Systems	Type, Position
ACCIDENTS:	
Problems during excavation (e.g. face instabilities)	Event, Causes and Zone
Water inflow	Flow typology, Zone

Tab. 4.4. Geological and Hydrogeological Information.

Geological and Hydrogeological Conditions	**Data**
ROCK MASS:	
	Rock mass type
	Lithology
	Geological Profile
GROUND WATER:	
	Ground water position with respect to the tunnel axis (Water pressure)

The "Environmental and operational information" section (as summarised in Table 4.5) includes the present factors as traffic, loads and environmental/surroundings conditions that may influence tunnel deterioration and future behaviour. According to [30], road tunnels are exposed to a wide range of aggressive chemicals that deteriorate concrete. The variety of chemical attacks depends on several factors as material quality, ground water composition and operational conditions. For example, temperature affects concrete and rock mass by thermal expansion and contraction and by freezing-thawing cycles. Frozen water (both from environment moisture and ground water) induces internal stresses and consequent cracking and/or spalling. Moreover, according to [59], this process is a catalyst for de-icing salts corrosion: as a matter of fact, an increase in lining permeability increases Chlorides penetration speed in the concrete lining.

Tab. 4.5. Environmental and operation information.

Environmental and Operation Conditions	**Data**
ENVIRONMENT:	
	Temperature and Ambient Moisture
	Aggressive ground water (chemical composition, flux)
OPERATION:	
	Aggressive Atmosphere (ventilation system)
	Traffic conditions
	De-icing Salts

The "Maintenance information" section (see Table 4.6) groups data about conservation procedures. Using information from inspections, maintenance and repair activities, it's possible to identify tunnel pathologies, follow their evolution, determine their causes and recognise possible recurrences. Survey data, for example, are particularly useful as disorder identification may reveal typical tunnel pathologies (see Chapter 2). Moreover, data from investigation (e.g. geophysical prospection by means of Ground Probing Radar or Thermal Thermography) inform about ground/support interface conditions, while core boring and laboratory tests allow the evaluation of both lining and rock mass mechanical properties during tunnel service life.

Tab. 4.6. Maintenance information.

Maintenance Conditions	**Data**
SURVEY ACTIVITIES:	
Monitoring	Type of measurement, frequency, monitored area
Inspection	Data and diagnosis (i.e. damages description, recognized pathologies)
Investigation	Type, place
OPERATIONS:	
Ordinary Maintenance	Type of intervention, frequency
Repair / Rehabilitation	Type of intervention, cause, frequency, place
Renewal / Refurbishment	Type of intervention, cause, date, place

The pathologies that affect crown and walls are different compared to the problems shown by invert, pavement, gutter and drainage system. Thus, in the technical form, for each kind of identified disorder it should be indicated:

- the type
- the location, extension and severity
- the possible causes and development scenarios (i.e. observation date and estimated evolution rate).

17 typical tunnel disorders, have been selected based on a detailed literature review (see Chapter 2), on information collected during Swiss Road tunnels inspections, and on considerations summarised in the previous paragraphs. Among the identified disorders type (reported in Table 4.7), many are typical of concrete structures. Indeed as National Roads network developed a lot during and after the 1960s, all tunnels are characterised by a concrete or, in some cases, a reinforced concrete lining. It is finally necessary to point out that all the disorders that can affect the electromechanical equipment are not taken into account in this work.

Tab. 4.7. Typical road tunnels disorders.

Type	**Disorder**
[1]	Water leakage, Moisture
[2]	Efflorescence (sulphates)
[3]	Staining, Calcium leaching effects, Calcareous concretion, Honeycomb
[4]	Concrete spalling, Delaminated concrete (due to corrosion of reinforcements)
[5]	Plain concrete corrosion, Concrete scaling (de-icing salts attack)
[6]	Corrosion of steel bars (reinforced concrete)
[7]	Voids behind the lining
[8]	Cracks, Fissures
[9]	Local deformation (crown)
[10]	Local deformation (walls)
[11]	Fines transport in the drainage system
[12]	Ice formation
[13]	Concrete lining crumbling - local failure, blocks fall
[14]	Track scaling
[15]	Invert heave up
[16]	Impact damages
[17]	Drainage system obstruction by calcareous concretion

Probably because the technical form requires quite detailed information, and its filling is time consuming, only few cantonal administrations contributed to the data collection: Geneva, Schaffhausen, Schwyz, St.Gallen and Vaud. The majority of data was collected directly by LMR (F. Sandrone and J.-F. Mathier) consulting cantonal archives for road infrastructures. Figure 4.2 represents how the data were obtained for each canton. Moreover, as the information comes from several sources, Table 4.8 summarises for each canton the quality of the collected information compared with the original source, the FGU-SIA tunnel inventory. In Appendix I two examples of technical form are provided: one filled directly by cantonal administration and another one filled by LMR.

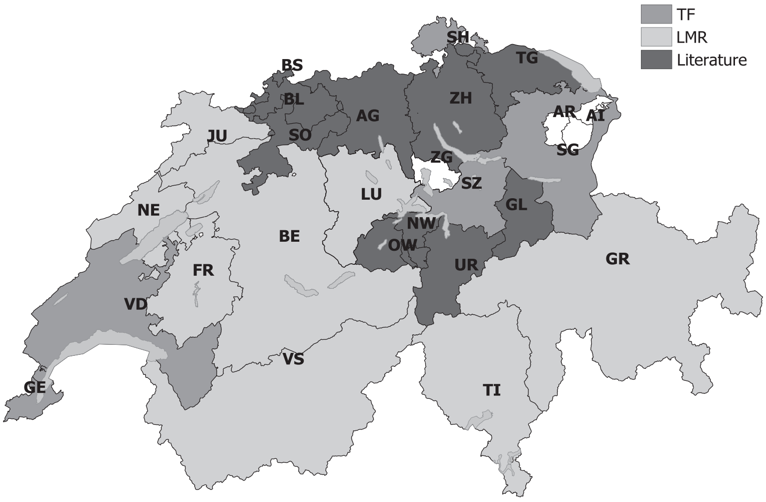


Fig. 4.2. Information sources, for the different cantons. In white the cantons without any National Road tunnel.

Tab. 4.8. Information type and quality: (-) poor quality, (+) medium quality, (++) good quality and (+++) very good quality.

Information Source	**Cantons**	**Quality of Construction Information**	**Quality of Operation / Environmental Information**
FGU-SIA Tunnelstatistik	All	+	-
Technical form filled directly by cantons	GE, SG, SH, SZ, VD	++	++
Technical form filled by LMR	BE, FR, JU, GR, LU, NE, TI, VS german part	++	+++
Literature (main sources: Tunnelling Switzerland, Strasse und Verkehr)	AG, BL, NW, OW, SO; TG, UR, ZH	++	-

4.4 Data Base

The data base has been created using a specific software for data collection and treatment (i.e. Access part of Microsoft Office Professional ed. 2003). At present, the Swiss Tunnel Data Base (TDB) stores detailed information of 168 tunnels. Figure 4.3 shows for each canton the number of tunnels recorded in the data base (i.e. empty bars) and the number of tunnels for which it has been possible to collect also maintenance information (i.e. black filled bars, total 122). Tunnels that pass through two cantons have been considered only as belonging to the canton responsible for management and conservation (e.g. Gotthard Tunnel belongs to both Canton of Uri and Canton Ticino but only Canton of Uri is responsible for tunnel management, thus this tunnel is considered as belonging to Canton of Uri).

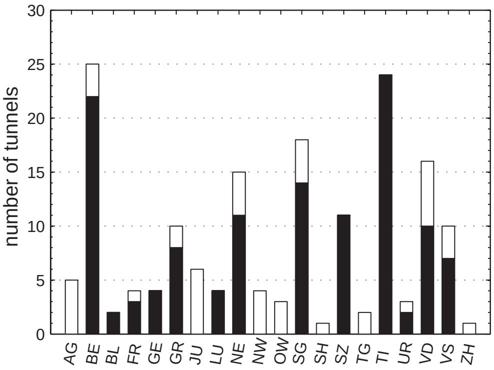


Fig. 4.3. Empty bars represent the number of tunnels recorded in the Tunnel Data Base for each Canton (i.e. tot 168), while black filled bars represent the number of tunnels with detailed information about maintenance operations (i.e. tot 122).

The data base main structure reflects the technical form. Apart from general information, it is possible to identify three main sections of data (see Table 4.9 [128]):

1. Construction,
2. Environment and Operation,
3. Maintenance.

Tunnels are identified by means of a specific code (i.e. ID number) and several tables are used for each data section. Moreover, in order to maximise the data consistency between tunnels in the data analyses, for each factor (i.e. variable) several modalities/attributes have been identified and coded as it is detailed in the Appendix II. This could be done after a detailed bibliographical research improved by tunnel inspector's advices (e.g. people from CETu, France and Perss Ingénieurs conseils SA Fribourg, Switzerland). An important feature that characterises the TDB, as described in Table 4.9, is the heterogeneity of collected information. Two typologies of data are used to describe each tunnel:

- Qualitative information (e.g. pathology description, geology, etc.), and
- Quantitative information (e.g. structural and geometrical data).

Tab. 4.9. Swiss Tunnel Data Base Structure. [128]

Section	Data
General Information	Tunnel name
	Town, Canton
	Road
	Local Operator
	Commissioning (operation) year
	Coordinates X,Y (centre point)
	Lane No.
Construction Information	Construction Year
	Geometrical Data (depth, length, section size, interaxis)
	Excavation method
	First support (type and length along the tunnel)
	Definitive lining (type, thickness and length)
	Waterproofing and drainage
	Accidents during construction
	Geological profile and description
	Geological difficulties during excavation
Environment and Operation Information	Accidents during operation
	Traffic
	Temperature
	Humidity
	Chemical composition of tunnel atmosphere
	Chemical composition of groundwater
	Groundwater level and circulation type
	Technical equipment (ventilation)
Maintenance Information	Inspection (date and frequency)
	Monitoring
	Routine maintenance
	Disorders (date of observation, possible cause, area and eventual repair)
	Renewal /Refurbishment (intervention date and type, area, cause)

4.4.1 G.I.S. tools

By means of an appropriate software (i.e. Manifold System 6.50 by Manifold Net LTD), using the (X, Y) coordinates of the centre point of each tunnel, the information collected in the Swiss Tunnel Data Base is associated with a spatial definition. This allows representing on a geographic model (e.g. a map) the actual conditions of each tunnel and provides the possibility of analysing the information by means of G.I.S. tools. An interesting feature typical of G.I.S. tools is the possibility of integrating several sources of information. By superposing Tunnel Data Base information on G.I.S. data as digital terrain model (DTM), geotechnical and hydrogeological maps, it is possible to provide (at a very general level) complementary information and supply to lacks of collected data, especially about geological conditions, at the tunnel scale. Together with the information recorded in the Tunnel Data Base and georeferenced by means of (X, Y) coordinates of each tunnel, it has been possible to integrate information from several sources:

1. National Roads network (sources: StradaDB, OFROU - Federal Roads Office, and Vector 25, Swisstopo - Federal Office of Topography),
2. Digital terrain model (source: MNT25, Swisstopo - Federal Office of Topography),
3. Geotechnical map (source: OFEV - Federal Office for the Environment),
4. Hydrogeological map (source: OFEV- - Federal Office for the Environment),
5. Traffic information (sources: OFROU - Federal Roads Office, and OFS - Federal Statistical Office),
6. Meteorological data (source: MétéoSuisse - Federal Office of Meteorology and Climatology).

Figure 4.4 and Figure 4.5 show examples of representation of National Roads tunnels (i.e. small gray circles) recorded in the data base on G.I.S. maps. The possibility of improving information by means of G.I.S. maps, though interesting, has to be considered reliable only for general statements at the land (road network) scale. Moreover, the use of superficial and shallow geotechnical and hydrological information is allowed only by keeping in mind that the majority of Swiss National Roads Tunnels is about 80-100 m deep [127], as already shown in Figure 4.1. Based on these considerations, G.I.S. tools have been used for preliminary detection of potential pathologies and typical pathology scenarios elaboration (see Chapter 5). As it is possible to see, some tunnels (i.e. small circles, in Figure 4.4 and Figure 4.5) are not placed on the roads network represented in the map. This is due to the fact that those tunnels belong to third class National Roads which are not represented in the StradaDB (source: OFROU).

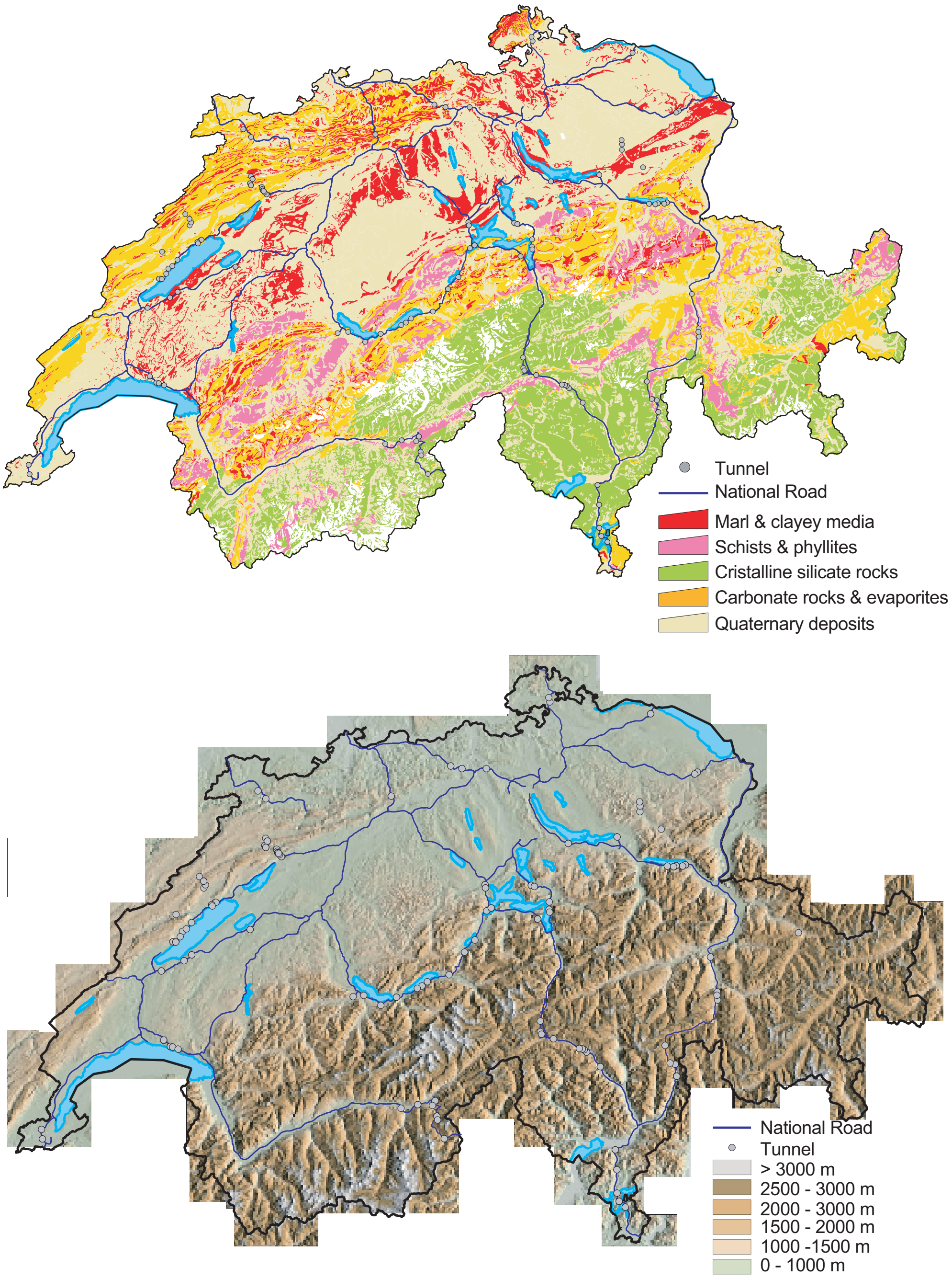


Fig. 4.4. G.I.S. Representation of tunnels on the National Roads network (blue lines, source: OFROU) and superposition of several layers. Top: geotechnical map (source OFEV); bottom: DTM (source: MNT25, Swisstopo). Tunnels (i.e. small gray circles) are identified by the X,Y coordinates recorded in the Tunnel Data Base.

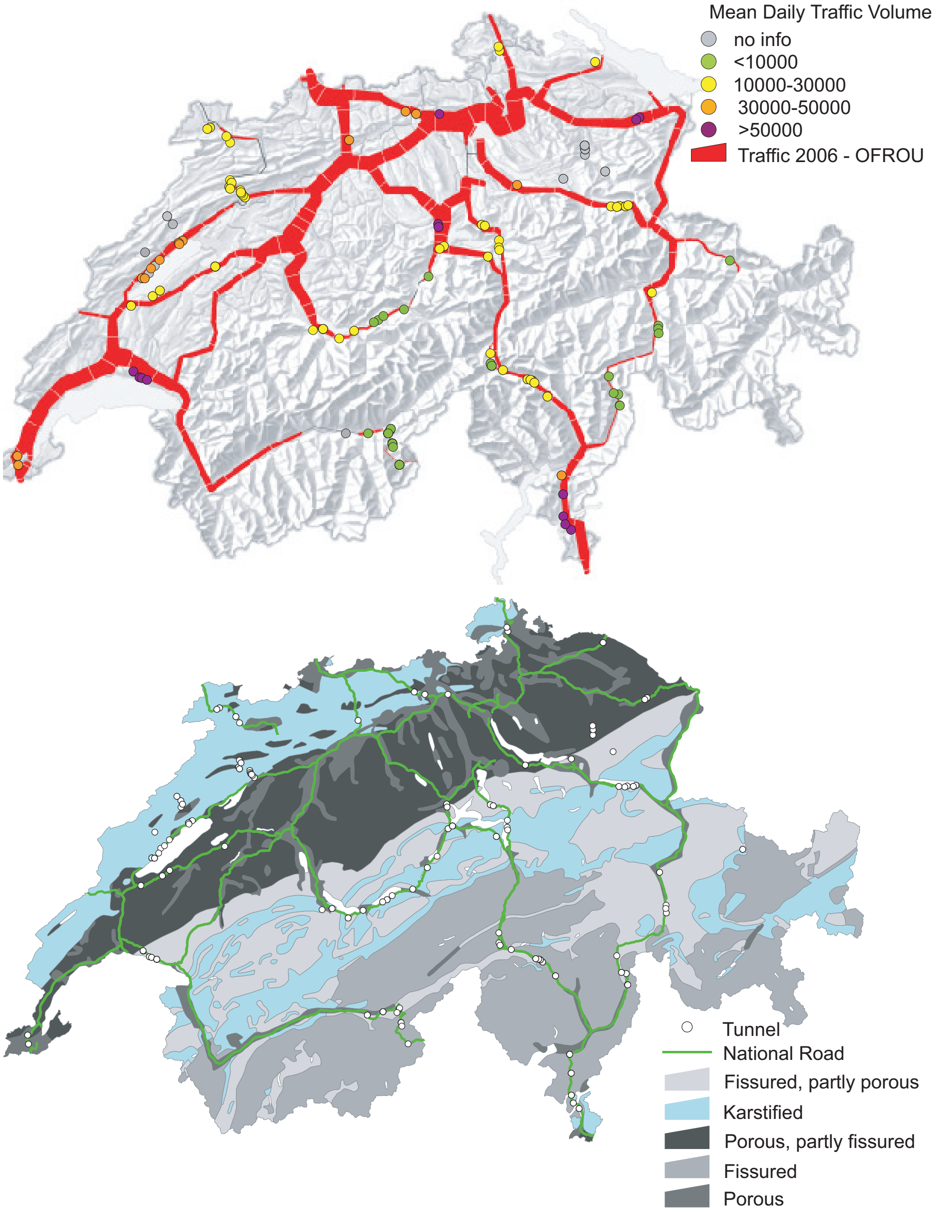


Fig. 4.5. G.I.S. Representation of tunnels on the National Roads network (green lines, source: OFROU) and superposition of several layers. Top: Traffic data (source: OFROU, 2006); bottom: hydrogeological map (source: OFEV). Tunnels (i.e. small circles) are identified by the X,Y coordinates recorded in the Tunnel Data Base.

4.5 Conclusion

The disorder typologies are influenced by the age of the tunnel [30] which determines the construction method and the time effects, and by the whole operation which comprehends environmental conditions and maintenance. Knowing the construction method can be useful in estimating the ground conditions around the excavation and the potential extension of the disturbed area around the tunnel [89]; [20]. Moreover, lining materials quality can decrease due to aggressive environment. With this information, the tunnel owner should be able to make a diagnosis of the tunnel conditions, suggesting eventual repairs. According to [98], evaluate tunnels conditions and predict their long term performance are the most difficult tasks for a tunnel owner. An evaluation of tunnel global conditions in terms of safety, serviceability and durability may require a detailed analysis of each part of the lining structure and the excavated rock mass. Collect and store up-to-date information about tunnels, after their construction and during operation, may help the tunnel owner to better evaluate the actual behaviour of the structure, follow its evolution and identify the possible origins of pathologies. Moreover, this can improve also planning of maintenance and repair activities during tunnel service life.

The Swiss Road tunnel data base (TDB) may be considered as an effective tool for providing a general description of the Swiss National Roads tunnels conditions. Moreover, it aims at term to become a management tool. It collects information related to the whole tunnel life. By means of G.I.S. tools, the data can be represented and visualised on a geographic model (i.e. a map). Another advantage of using G.I.S. tools, under some conditions, and when there's lack of data, is the possibility to supply information by superposition of appropriate layers (e.g. digital terrain model, geotechnical and hydrogeological maps) [127].

5. TDB Analysis

The Tunnel Data Base, presented in Chapter 4, regroups information of different type and nature, related to the whole tunnel life. After collecting data about tunnel characteristics and observed disorders, it is necessary to identify main pathologies causes. Information about the main disorders that affect road tunnels, together with detailed data about tunnel construction, operative environment and maintenance procedures are necessary for identifying typical pathologies.

In the first part of this chapter a general portrait is provided, based on the main features of Swiss National Roads tunnels. Based on their initial conditions, structures are described in terms of their degradation potential. The main factors that determine the tunnel initial conditions and its degradation potential are identified. Then, the chapter focuses on the degradation rate of tunnels during their service life, based on the data collected from the tunnel inspections and on the literature review presented in Chapter 2. Influence factors that determine disorders development are analysed. Combining G.I.S. tools (preliminary detection) and multivariate statistical approach, the main factors that do influence long term deterioration are pointed out and described. Finally, representative pathology scenarios are identified.

5.1 National Road tunnels portrait

The data collection presented in Chapter 4 allows characterising and describing tunnels by means of selected features. Preliminary statistical analyses represented in Figure 5.1, Figure 5.2 and Figure 5.3 outline a comprehensive portrait of Swiss National Roads tunnels collected in the Tunnel Data Base.

In order to standardise information and facilitate data collection and treatment, each tunnel feature has been detailed with a specific range of attributes (i.e. modalities). Both qualitative and quantitative attributes, used for characterising each tunnel were selected after an accurate literature review together with advices from tunnel inspectors (e.g. CETu, France and Perss Ingénieurs conseils SA Fribourg, Switzerland). For each selected feature, statistics show how the whole data base population is distributed through feature modalities/attributes. Two kinds of representation have been chosen:

- quantitative features are described by means of bar plots (Figure 5.1),
- qualitative features are described by means of pie charts (Figure 5.2 & Figure 5.3).

Moreover, these statistical analyses show also the distribution of the “inspected tunnels population” (i.e. tunnels for which principal inspection data have been collected) which is smaller than the total number of tunnels recorded in the data base (i.e. respectively, 122 vs. 168 tunnels). In Figure 5.1 (quantitative features), both tunnels populations are represented by, respectively, empty vs. filled bars. In Figure 5.2 and Figure 5.3 (qualitative features), the pie chart represents the total data base population (i.e. [TDB]), while the “inspected tunnels population” is only represented by labels (i.e. [PI]).

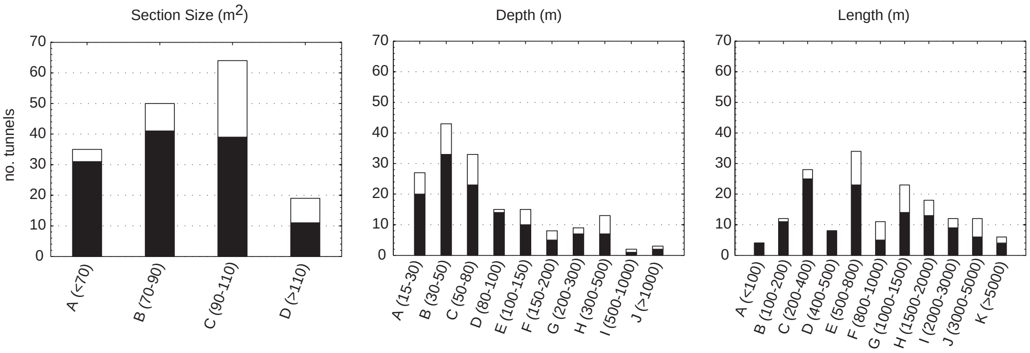


Fig. 5.1. National Roads tunnels quantitative features. From the left: tunnel section size (m^2), depth (m) and length (m). Empty bars represent the total number of tunnels (i.e. the whole population of the Tunnel Data Base), while black filled bars represent the number of inspected tunnels (i.e. tunnels for which it has been possible to collect data about principal inspections; see also Figure 4.3, Chapter 4).

Section size. The tunnel section size is ruled by norms [135], i.e. according to the road type and on the number of lanes, together with traffic conditions, a specific gabarit is required. As it is possible to see (Figure 5.1), the majority of National Roads tunnels has a section around 90 m^2. However, the section size can be smaller (i.e. about 70 m^2) or bigger (i.e. about 110 m^2) depending on the cross section shape and, as it will be shown in the following, on the construction method.

Depth. The maximum overburden is considered in the data analysis. This simplification has been done by observing that:

- for many National Roads tunnels, the maximum depth is rather equivalent to the mean depth,
- for short tunnels the whole tunnel is usually designed for bearing the highest load, which corresponds to the worse conditions.

For a majority of tunnels it has been possible to get the depth information directly from the technical form. However, for about 20 tunnels, due to lack of data, it has been necessary to draw the depth profile using G.I.S. tools (superposing of the tunnel position on a Digital Terrain Model from Swisstopo) and, then, to determine the maximum overburden. From the depth distribution drawn in Figure 5.1, three main depth classes can be considered:

- most tunnels are less than 80 m deep,
- a certain number of tunnels is between 80 and 500 m,
- very few tunnels are more than 500 m deep.

Length. In terms of length, the data base population could be split into two: about 86 tunnels are shorter than 800 m, while the rest is longer up to more than 5000 m. The majority of the population has a length between 500 and 800, though a remarkable number of tunnels are shorter than 400 m. Finally, several tunnels are between 1000 and 1500 m long. As it will be shown in the following, tunnel length, coupled with information about traffic volumes, ventilation system and use of de-icing salts, seems to be quite important in explaining lining weathering pathologies.

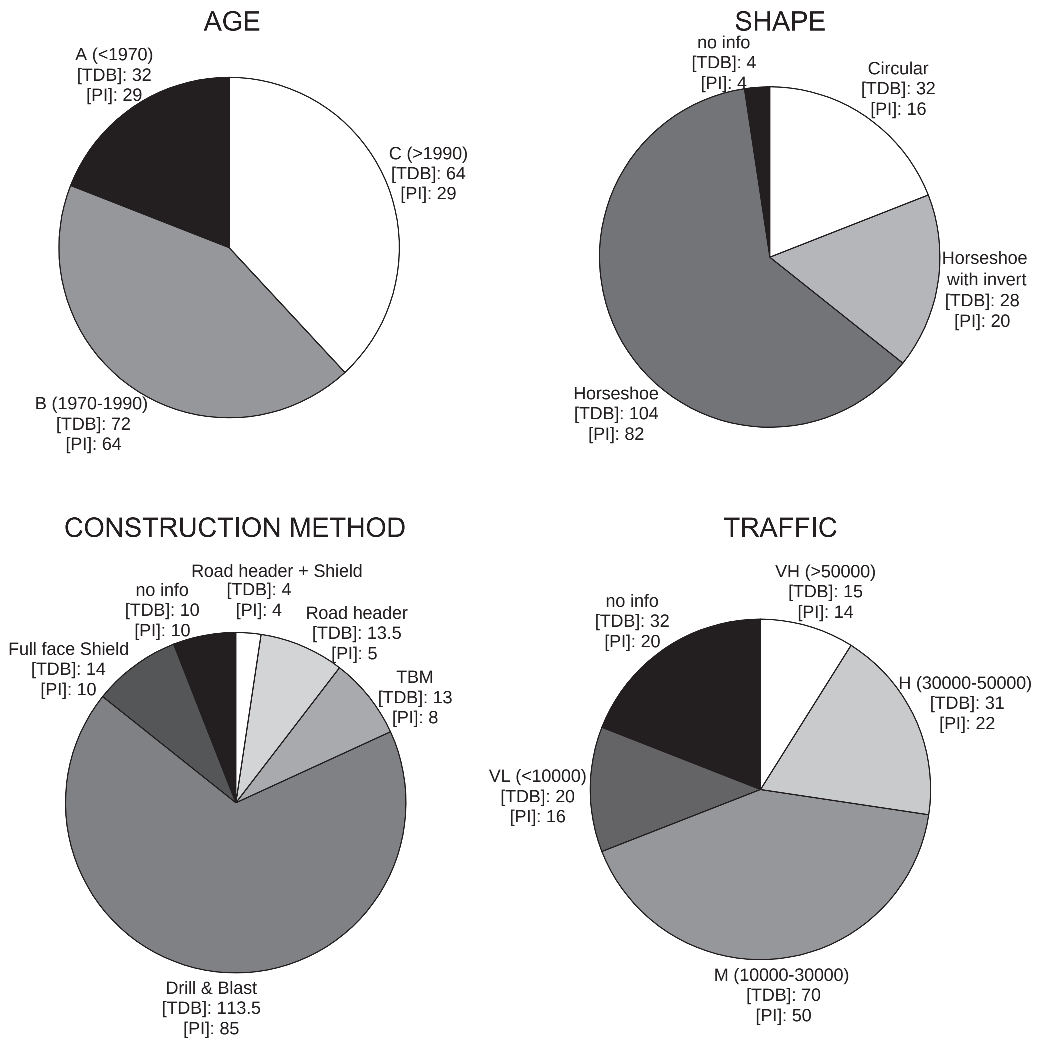


Fig. 5.2. Tunnels features statistics 1. For each modality/attribute represented in the pie chart it is specified: [TDB] the number of tunnels recorded in the TDB and [PI] the number of inspected tunnels (i.e. for which there's information about the last principal inspection). The traffic values are expressed in (vehicles/day).

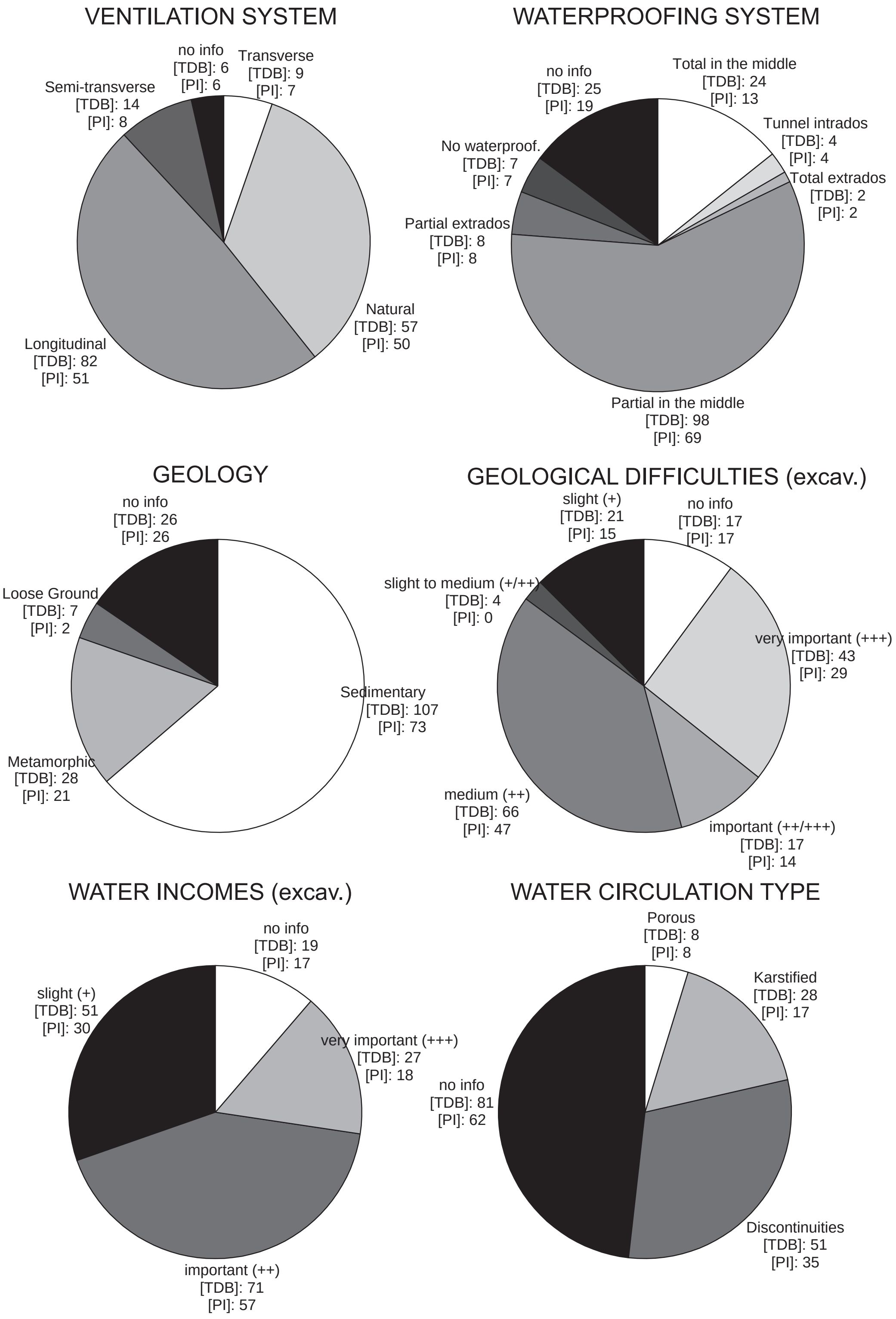


Fig. 5.3. Tunnels features statistics 2. For each modality/attribute represented in the pie chart it is specified: [TDB] the number of tunnels recorded in the TDB and [PI] the number of inspected tunnels (i.e. for which there's information about the last principal inspection).

AGE. Classifying tunnels by their age means also taking into consideration construction techniques evolution. By combining these two features and by considering operation as an active cause in degradation phenomena, the age of the tunnel has been calculated in terms of service life duration, starting from the commissioning year (operation). The Swiss Road Tunnels can be divided into 3 main classes (as shown in Figure 5.2):

A. Old tunnels: constructed before the year 1970. Those tunnels can be affected by ageing of lining and other problems depending on concrete quality and construction techniques. Often, old tunnels need renewal procedures in order to respect new safety and operational requirements. Operating for about 40 years, this class of tunnels represents the 19% of the population of the data base.
B. Rather old tunnels: excavated between the years 1970 and 1990. Though characterised by a better quality of materials and construction techniques if compared to tunnels of class A, these tunnels start showing long term pathologies. This class represents about the 43% of the total population.
C. Recent tunnels: constructed after the year 1990. In this class, under normal conditions (i.e. without evident/major construction defects/errors), the tunnels don't show any problem due to ageing. Operating for less than 15 years, this class of tunnels represents about the 38% of the total population of the data base.

For each age class it has been evaluated, according to the collected data, the percentage of tunnels that has been inspected. The majority of tunnels constructed and commissioned before the nineties has been inspected during the last two years (i.e. 13 of class A and 28 tunnels of class B inspected in 2005 and 2006). However, no information was found on the last principal inspection of a certain number of tunnels, especially the recent ones (i.e. about 50 tunnels commissioned after the year 1990).

GEOLOGY. For what concerns geology, three main categories of excavated rock-ground mass have been identified:

- Sedimentary,
- Metamorphic,
- Loose ground.

As shown in the geology pie chart (Figure 5.3) the majority of tunnels has been excavated in sedimentary rock mass as calcareous sandstone, limestone, marl and clay. More than 15% of tunnels crosses metamorphic rock masses that include schists and phyllites but also gneiss of the Alps. For a portion of the data base population it wasn't possible to collect any geological data directly by means of the technical form (Section 4.3). G.I.S. geotechnical data were used for supplying this lack of data. Anyway, though this operation allows to improve collected data with additional information, it is important to keep in mind that it represents roughly the probable geology. Indeed, even if the tunnels are rather shallow, the geology at the tunnel depth is not necessarily similar to the geology observed in surface, reported in the G.I.S. map. Moreover, the precision and the scale of the G.I.S. document are not comparable at all to those of geological data compiled during the tunnels construction.

Apart from quaternary deposits, the geotechnical classes of the Swiss 1:200'000 vector map (source: OFEV - *Carte géotechnique simplifiée de la Suisse*) were merged in five principal classes:

- Marl and clayey media (S_cm),
- Carbonate rocks (S_ce)
- Evaporites (S_ce+),
- Schists and phyllites (M_sp),
- Crystalline silicate rock masses and gneiss (M_g).

Considering this classification, the distribution of the tunnels population is represented in Figure 5.4. For tunnels that cross more than one type of rock mass a contribution proportional to the excavated length in each class has been considered.

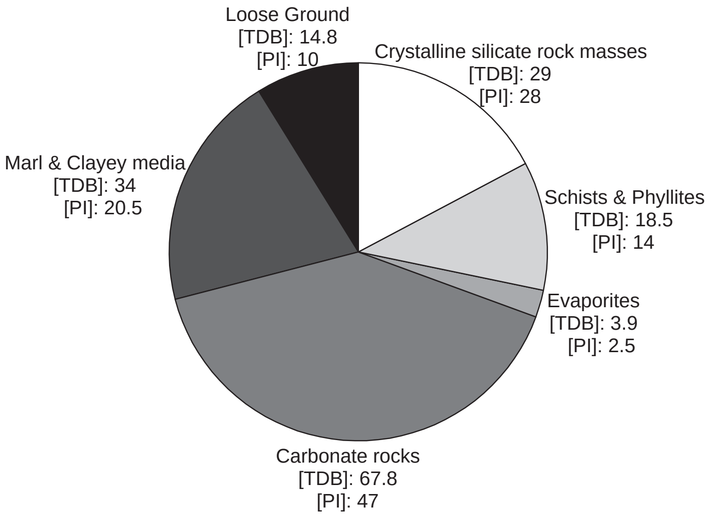


Fig. 5.4. Tunnels geological conditions statistics determined from G.I.S. geotechnical data (source: Carte géotechnique simplifiée de la Suisse - 1:200'000 - OFEV). For each class is represented: [TDB] the number of tunnels recorded in the TDB and [PI] the number of inspected tunnels (i.e. for which there's information about the last principal inspection).

In the data base, geological information is completed by considerations about geological difficulties encountered during excavation together with hydrological information and water incomes during excavation. All those data are represented by pie charts in Figure 5.2. Mainly based on geotechnical considerations, some attempts were done to identify relationships between the difficulties encountered during excavation and the geological formations. Probably due to the qualitative and subjective nature of data, no relation was found. For this reason, this information will not be taken into consideration for further analyses.

Together with the construction year, also the geological conditions may determine the construction method. As represented in Figure 5.2, the majority of tunnels (i.e. 67.5%) has been excavated with traditional methods. Full face excavation is more recent and characterises only about the 17% of the data base population (considering TBM and Shield together). This information can be linked with the section size described in Figure 5.1 and shape in Figure 5.2. The majority of tunnels has a horseshoe shaped section of about 80 m^2 that corresponds to the average value for tunnels excavated with traditional methods. Bigger section size (i.e. about 100 m^2) corresponds to circular shape tunnels, generally excavated by full face tunnel machines (i.e. TBM and Shield).

HYDROGEOLOGICAL CONDITIONS & WATERPROOFING SYSTEM. For what concerns hydrogeological information, as it has been done for the geological conditions, G.I.S. information has been used to supply lack in collected data. The classification proposed by the hydrological vector map (Figure 4.5 bottom; source: OFEV) was considered to describe the water circulation around the excavation:

- about 49% of tunnels crosses discontinuous (i.e. fissured and partly fissured porous rock masses, with water circulation through discontinuities),
- about 19% of tunnels crosses porous rock masses,
- 32% of tunnels crosses karstified formations.

Within the collected data, information about water inflow during tunnel excavation and waterproofing system should be related to the hydrogeological conditions. Water incomes during construction have been divided into three main classes: slight (+), important (++) and very important (+++). As it has been observed for the geological difficulties record, also in this case the information is too qualitative and the lack of detailed information (e.g. at which tunnel meter significant water inflows were observed) only allows making general statements. The majority of tunnels with important and very important water incomes are located in discontinuous and karstified rock masses. It is, however, important to note that for tunnels excavated in karstified formations a high percentage (i.e. about 30%) of water incomes is classed as slight, obviously related with the subjective nature of the information. This may be due to the fact that big karsts are found during geological investigation before construction or, also, to the fact that in the case of karsts water incomes are more local (limited to some tunnel meters) than in the case of discontinuous rock masses.

Apart for a small number of old tunnels without waterproofing system, or characterised by an internal waterproofing system placed at the definitive lining intrados (i.e. Tunnel intrados, Ti), it is possible to identify two main types of waterproofing in the Swiss National Roads tunnels (Figure 5.3):

- the partial system (i.e. crown and side walls) is typical of traditional excavation techniques and mainly used for tunnels in fissured and karstified rock masses. In the majority of tunnels it is placed between the support and the definitive lining (i.e. Partial in the middle, Pm), but it can be placed also in direct contact with the excavated rock mass (i.e. Partial extrados, Pe).
- the total system (i.e. crown, side walls and invert) is mainly associated with circular section and full face mechanical excavation in porous formations. As it happens for the partial system, in the majority of cases it is placed in between the support an the definitive lining (i.e. Total in the middle, Tm) but in few cases it can be in direct contact with the rock mass (i.e. Total extrados, Te).

OPERATION CONDITIONS: TRAFFIC & VENTILATION SYSTEM. As mentioned above, together with tunnel age and construction conditions (i.e. geology, hydrogeology and difficulties encountered during excavation), also operation contributes to tunnel degradation. Concrete lining deterioration may be a consequence of the aggressive atmosphere inside the tunnel which depends on several factors as tunnel length, ventilation system and traffic conditions. According to [145], it is possible to divide tunnels into four classes based on the Average Daily Traffic Volume (i.e. DTV, data for the year 2006 from OFROU):

- 12% of the total population of the data base is characterised by very low traffic: $\dot{D}TV \leq 10'000$ vehicles;
- 42% is characterised by a medium traffic: $DTV = (10'000) \div (30'000)$ vehicles;
- more than 18% has high traffic conditions: $DTV = (30'000) \div (50'000)$ vehicles;
- 9% is crossed daily by a very high traffic: $DTV > 50'000$ vehicles;
- for the last 19% (i.e. 32 tunnels) there’s no available information.

Short tunnels (i.e. less than 700-800 m long) are characterised by natural ventilation or in case of high and very traffic by longitudinal ventilation. Excluding about the 40% of the data base population composed by those tunnels with natural ventilation, the majority of structures is equipped with a longitudinal ventilation system (more than 48%) and the rest is equally divided between transverse and semi-transverse systems (see Figure 5.3). Ventilation system mainly depends on tunnel length and daily traffic, thus any relationship between those three factors has been investigated. Tunnels of a medium length (i.e. between 800 m and 2000 m long) have generally a longitudinal ventilation system that, for few cases with very high traffic conditions, changes into transverse ventilation. While, long tunnels (i.e. more than 2000 m long) have a transverse or semi-transverse ventilation.

The general statistics, described in this paragraph, provide information about construction features and operation conditions of Swiss National Roads tunnels (e.g. age, depth, shape of the tunnel, construction method, waterproofing and ventilation systems, traffic volumes and tunnel length, but also geological and hydrogeological conditions).

5.2 TDB exploratory examination

After characterisation of main tunnel features, a general exploratory analysis using all the collected information has been tried first in order to see whether any relationship between disorders and tunnel features could be easily pointed out. Anyway, mainly due to the high number of variables (i.e. number of disorders = 15 × number of influence factors = 15, each of them × number of attributes/modalities, thus more than 1000!) it was not possible to get any interesting result by such kind of analysis, thus, another approach has been chosen.

As written in Chapter 3, the Swiss National Roads Authority uses a specific data base, called KUBA-DB, for collecting maintenance information about road infrastructures (except tunnels). An optional tool, called KUBA-MS [102], was introduced first in Canton Ticino, in 1998, in order to evaluate bridges degradation starting from principal inspections observations. The data analysis process presented in this chapter is structured as follows (Figure 5.5):

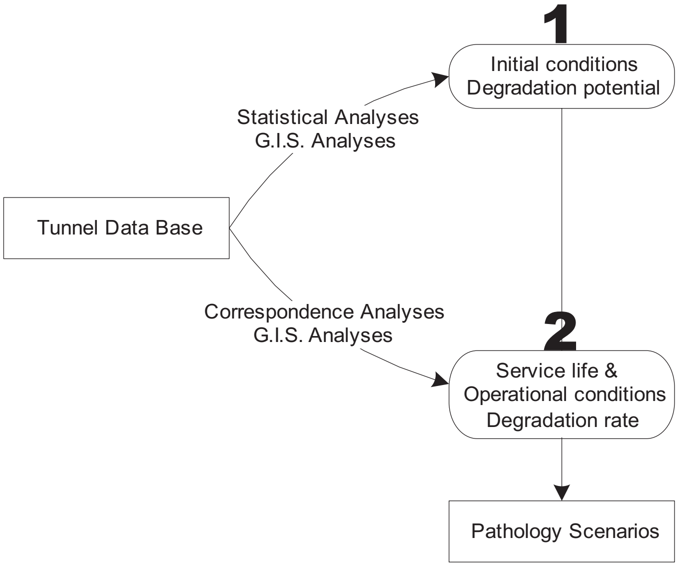


Fig. 5.5. TDB Analysis process: 1. initial conditions for investigating degradation potential and 2. service life conditions for investigating degradation rate. The arrays show the tools used for the analyses.

1. Evaluation of expected and unexpected influences of the different factors modalities/attributes on the tunnel initial conditions and degradation potential. The main tunnel features, described in the National Roads tunnels portrait, are considered as potential influence factors. In this first analysis step (see Paragraph 5.2.1), the Tunnel Data Base is analysed through frequency tables (i.e. contingency tables, Pivot Analysis in both Microsoft Access and MS Excel Spreadsheet) in order to evaluate the distribution of categorical variables (i.e. modalities) in the recorded sample. Moreover, combined with preliminary detection of potential degradation scenarios by means of G.I.S. tools, this method allows to point out, at a very general level, evident and/or systematic pathology development scenarios related to tunnel initial conditions (i.e. construction features, geology and other characteristics that depend on tunnel geographical location, as depth, for example).
2. Investigation of relationships between disorders type (see Table 4.7 in Chapter 4) and each factor modalities/attributes (or combination of factors). Evaluation of the degradation rate depending on the selected influence factors modalities/attributes. In the second analysis step, the influence of tunnel service life/operation conditions on the degradation speed/rate is investigated. Multivariate statistics tools as the Correspondence Analysis (more details about this method can be found in [60]; [92]; Nishisato, (2004), see [126]) are useful to describe dependencies between identified pathologies (i.e disorder type) and tunnel operational features (i.e. influence factors

modalities/attributes, for example, traffic, ventilation system...). These analyses take into account disorders recorded during the last principal inspection (see Paragraph 5.2.2). Finally, the disorder probability due to one or more influence factors modalities/attributes is evaluated (see Paragraph 5.2.3). This procedure is necessary to understand whether (or not) factors modalities identified by means of Correspondence Analyses have a major influence on the selected disorder development. Moreover, as in this case probability is evaluated by taking into account the number of affected tunnels, it is possible to identify representative pathology development scenarios.

Actually, the KUBA-MS procedure has a third step for simulating pathologies evolution. In this step, Markov Chains use probability values for describing decreasing conditions and predicting long term condition. The simulation can be done by taking into account the actual conditions of the structure (i.e. Present Conditions Matrix) and using evolution models based on successive inspections results. Due to a lack of information this is not possible for the moment with the National Roads tunnels. As already explained in Chapter 4, there isn't enough information (i.e. detailed information, at least) about successive inspections of the same disorder. Thus it is not possible to reconstruct disorders evolution by means of successive observations. To overcome this problem, degradation laws presented in Chapter 2 are used, in the next chapter, for predicting tunnel long term behaviour.

5.2.1 Tunnel initial conditions analysis

Tunnel degradation potential due to initial conditions is evaluated by following a simple procedure:

1. identify the influence factors that define the tunnel initial conditions within all the features collected in the data base, (see Table 5.1, column 1),
2. evaluate the influence of those selected factors on the tunnel degradation potential by calculating the mean number of disorders per tunnel for each feature modality class, (see Table 5.1, columns 2-3).

A better analysis of the influence of each factor on tunnel degradation potential would require a detailed description (i.e. meter by meter) of both construction conditions and disorders location. Actually, for the majority of tunnels in the data base, neither the geological and construction data nor the disorders observations during the principal inspections are sufficiently detailed to enhance the pathologies analysis. Calculating the mean number of disorders per meter of tunnel (i.e. dividing the total number of disorder by the length of the tunnel) instead of per tunnel, has been as well considered, but not adopted as it could result in misjudgements (e.g. if the disorders are concentrated in a certain portion of the tunnel). In the following, the contribution of each influence factor modality to tunnel degradation potential is evaluated from a function of the mean number of disorders per tunnel per influence factor modality class. In order to avoid data redundancy, due to successive inspections of the same tunnel (i.e. repeated observations of the same disorder, if any), the total number of disorders has been calculated by taking into consideration only data from the last tunnel principal inspection.

Tab. 5.1. Influence of tunnel initial conditions on the degradation potential: influence factors have been selected within all the tunnel features collected in the TDB. For each factor modality, the average number of disorders per tunnel observed during the last inspection has been evaluated. The influence is represented as follows: +++ = (>4), strong influence; ++ = (3-4), moderate influence; + = (2-3), low influence; - = (<2) no influence. Most of the attributes are represented by a label, in brackets, that will be used for analyses and graphical representations in the following

Factor	Modality/Attribute	Mean number of disorder/tunnel	Influence
Construction/ Commissioning Year	<1970 (A)	4.3	+++
	1970-1990 (B)	4.0	++
	>1990 (C)	3.5	++
Construction Method	Full face shield (Sh)	6.2	+++
	TBM (TBM)	2.3	+
	Drill & Blast (DB)	3.9	++
	Road-header (Rh)	3.5	++
	Road-header under shield protection (RhS)	1.8	-
Tunnel Geometry - section size (m^2)	<70	3.3	++
	70-90	4.5	+++
	90-110	2.9	+
	>110	7.5	+++
Tunnel Geometry - section shape	Circular (K)	3.9	++
	Horseshoe (H)	3.9	++
	Horseshoe with invert (HR)	4.7	+++
Waterproofing system	No waterproofing (K)	5.6	+++
	Partial in the middle (Pm)	3.9	++
	Partial extrados (Pe)	3.0	+
	Partial intrados (Pi)	-	
	Total middle (Tm)	5.4	+++
	Total extrados (Te)	1.0	-
	Tunnel intrados (Ti)	5.3	+++
Depth (m)	<80	4.2	+++
	80-500	3.7	++
	>500	2.3	+
Geology	Sedimentary rocks (S)	3.2	++
	Loose ground (L)	4.8	+++
	Metamorphic rocks (M)	5.5	+++
Geological difficulties during construction	Slight (+)	1.9	-
	Medium (++)	4.2	++
	Medium to difficult (++/+++)	2.6	+
	Difficult (+++)	5.1	+++
Water incomes during construction	Slight (+)	2.6	+
	Important (++)	3.6	++
	Very important (+++)	5.8	+++

Factor	**Modality/Attribute**	**Mean number of disorder/tunnel**	**Influence**
Water circulation around excavation	Karstified (K)	2.5	+
	Porous (P)	6.9	+++
	Discontinuous (D)	3.8	++

Results of Table 5.1 should be analysed by considering classes of tunnels with the same initial conditions, but, as already explained, investigate tunnel population distributions in classes of combined factors may result in statistical analyses of very few populated classes. Thus, after identifying the main features that characterise tunnel initial conditions, dependencies between factors have been estimated by means of Correspondence Analysis. This method, described in detail in [60]; [92]; Nishisato, (2004) (see also [126]), allows investigating statistical associations between categorical variables (i.e. a variable having two or more categories/modalities without intrinsic order). In this way, a preliminary selection of features can be operated in order to reduce, where possible, the number of relevant variables (i.e. influence factors) involved in defining tunnel initial conditions. Table 5.2 summarises the Correspondence Analyses results. In order to check if results are plausible two analyses with obvious positive and negative results have been done:

1. Section shape is expected to be dependent on construction method, that corresponds with the analysis result (i.e. high dependency);
2. Construction year is expected to be independent from water incomes during construction as shown by the analysis result (i.e. no dependency).

It is quite interesting to see that, as anticipated in the previous paragraph, construction year influences construction techniques. Construction method influences tunnel shape and size and also the waterproofing system. It is quite surprising, instead, that the waterproofing system is less related to water incomes during construction but this result may depend on the fact that waterproofing system is mainly influenced by the construction year. Also depth seems to not influence neither the tunnel shape nor the waterproofing system. This can be explained with the fact that the shape of the tunnel section mainly depends on the excavation method and on the construction year.

Tab. 5.2. Tunnel main features relationships evaluated by means of Correspondence Analysis. Dependency values have been interpreted as suggested by specific literature: (-) no dependency = values less than 0.2; (+) slight dependency = values between 0.2 and 0.4; (++) medium dependency = values between 0.4 and 0.6; (+++) high dependency = values bigger than 0.6. The two analyses used to check results are written in italics.

Variable X	**Variable Y**	**Dependency value**	
Construction Year	Water incomes during construction	0.19	-
Construction Year	Section shape	0.55	++
Construction Year	Waterproofing System	0.62	+++
Construction Year	Construction Method	0.61	+++
Construction Method	Section shape	0.86	+++
Section shape	Waterproofing System	0.73	+++
Section shape	Section size	0.54	++
Construction Year	Section size	0.48	++
Construction Method	Section size	0.49	++
Construction Method	Waterproofing System	0.60	++
Water incomes during construction	Waterproofing System	0.35	+
Geology	Construction Year	0.33	+
Geology	Construction Method	0.42	++
Geology	Section shape	0.37	+
Geology	Waterproofing System	0.36	+
Depth	Section shape	0.17	-
Depth	Waterproofing System	0.39	+
Depth	Water incomes during construction	0.33	+

By analysing factors relationships (Table 5.2) and focusing only on the main representative features (see Figure 5.6), it is possible to reduce the analysis of tunnel initial conditions contribution to degradation to:

- commissioning year, construction method and equipment (i.e. waterproofing system),
- tunnel depth and geological conditions (including squeezing potential), hydrogeological conditions (i.e. water incomes during construction and water circulation around excavation).

As explained in Section 5.1, the number of tunnels for which it was possible to collect maintenance details does not correspond to the total population of the data base. Thus, this analysis was performed on only about 73% of the total number of tunnels in the TDB (i.e. 122/168). Moreover, to avoid data redundancy, the total number of disorders is evaluated by considering only data from the last principal inspection.

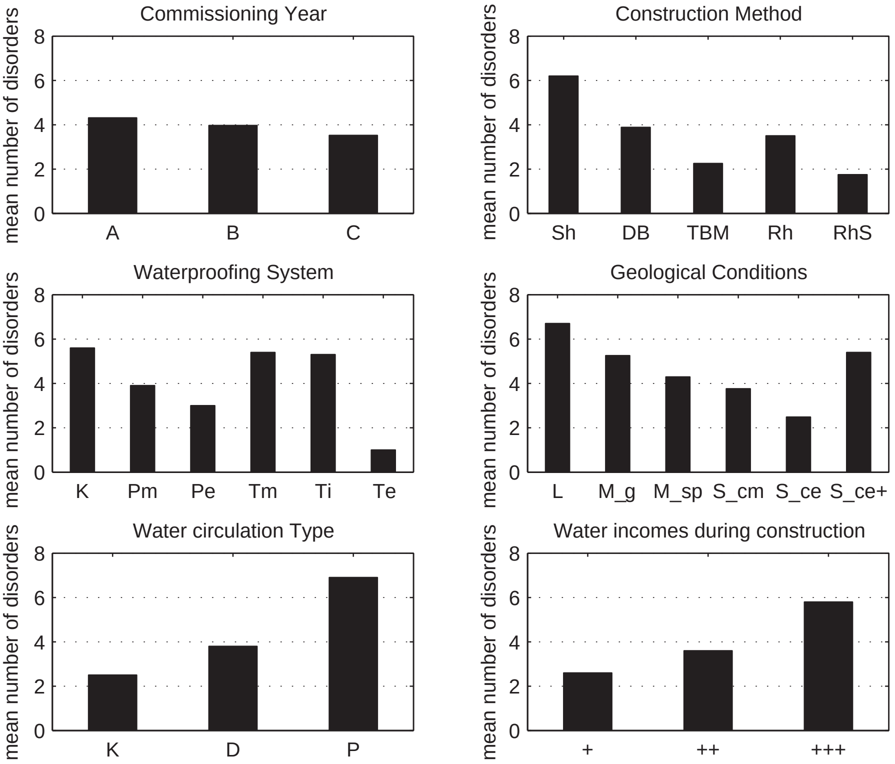


Fig. 5.6. Initial conditions influence on degradation potential. For each factor it is represented the mean number of disorders per tunnel per attribute class. Apart from geological conditions, labels are described in Table 5.1. Geological classes are described in Section 5.1: L = loose ground; M_g = gneiss; M_sp = schists and phyllites; S_cm = clays and marls; S_ce = calcareous rocks; S_ce+ = evaporites.

COMMISSIONING YEAR, CONSTRUCTION METHOD & WATERPROOFING SYSTEM. As expected, the mean number of disorders per tunnel, recorded during the last principal inspection, is slightly higher for tunnels constructed before the year 1970, if compared with recent tunnels. Considering the construction techniques the higher mean number of disorders characterises tunnels constructed under a shield protection, followed by tunnels excavated by drill and blast (i.e. traditional method). To better interpret this result it is important to take into account the total number of tunnels per class: the small population of tunnels excavated with full face shield can be considered less representative than the class of tunnels excavated with drill and blast techniques (i.e. the ratio of tunnels per class is 10 to 85!). This hypothesis is supported also by considering that, especially in the past, the use of explosives increased the size of the disturbed/damaged zone around the tunnel and increased deterioration due to weathering. Moreover, as discussed during ISRM, 2007 - Special Session S05: Maintenance and Repair of Underground Constructions, chaired by Prof. Ribeiro e Sousa, together with blasting, also the so called “Belgian Tunnelling Method”, an old excavation technique based on face partitioning, has a significant disturbing action on the excavation contour (i.e. it may increase the damaged zone extent). According to the collected data, this method has been used for about 33% of old and very old tunnels.

Another important result is the influence of the waterproofing system: as expected, when there is no sealing tunnels have a higher degradation potential. Also the presence of an internal waterproofing system is responsible of high degradation potential. However, Figure 5.6 shows how several disorders may also affect tunnels with total waterproofing system. Indeed, in absence of an adequate drainage system, tunnel disorders may be caused by additional loads development due to water pressure increase around the tunnel. This phenomenon will be studied more in detail in the following (see Chapter 6).

GEOLOGICAL & HYDROGEOLOGICAL CONDITIONS. As stated in Chapter 2, tunnel deterioration depends, among others, on geological and hydrogeological conditions. One of the bar plots in Figure 5.6 shows how the mean number of disorders changes due to geology. As explained in Section 5.1, the former three main classes of the data base (i.e. respectively, loose ground, sedimentary and metamorphic rock masses) become 6 based on their different geotechnical behaviour. Apart from loose ground (L), sedimentary rocks were split into calcareous rock masses (S_ce), evaporites (S_ce+) and marls and clays (S_cm), schists and phyllites (M_sp) have been separated from crystalline rocks (M_g). The higher number of disorders has been recorded for tunnels excavated in loose ground masses, probably due to the poor quality of this formation. Moreover, the significant number of disorder for tunnels excavated in marl and clayey media or schists and phyllites shows how time dependent pathologies of excavated rock masses may have a critical impact on tunnel stability (e.g. external load increase vs. insufficient lining bear capacity). Also the tunnels excavated in gneiss are characterised by a higher number of disorders, probably due to the fact that this formation is often rather blocky. Among the disorders identified during tunnel principal inspections (see Table 4.7 in Chapter 4), cracks, fissures and invert heave up, for example, can be caused by excessive deformations due to the surrounding rock mass delayed behaviour (e.g. creep, squeezing and/or swelling behaviour). Considering water circulation conditions around the tunnel, the mean number of disorders reported in Figure 5.6 shows how the contribution on tunnels degradation potential of porous rock masses may reveal quite important.

Nevertheless, also carbonate and evaporites are affected by a relevant mean number of disorders. As a matter of fact, compared to other geological formations, sedimentary rocks are particularly sensible to water physical and chemical actions. Thus, depending on ground water conditions, among the disorders identified during principal inspections, efflorescence, calcareous concretions and drainage system obstructions may be related to rock masses weathering potential.

Starting from the Swiss 1:200´000 geotechnical vector map, analyses were performed to detect tunnels potentially affected by rock mass pathologies. Results are shown in Figure 5.7, respectively:

- on top, zones with a high weathering potential (i.e. calcareous formations and evaporites);
- on bottom, geotechnical classes with a potential delayed behaviour (i.e. marl and clayey media, schists and phyllites).

As it is possible to see, marl and clayey media are concentrated in both the north-west and central part of Switzerland (i.e. Jurassic claystones and tertiary marls of Jura Mountains; Molasse Basin and Helvetic Nappes marls), while schists and phyllites characterise the Penninic formation of Valais, Ticino and Graubünden Cantons. On the contrary, calcareous rock masses can be found especially in the central part of Switzerland (Molasse Basin and Helvetic Nappes, as described in [144]).

Among the 168 tunnels in the TDB, the 122 ones with last inspection data have been checked in order to verify whether potential pathologies identified through G.I.S. analyses are in accordance with observed disorders. Moreover, this procedure may clarify if and how, within initial conditions, rock mass geotechnical characteristics contribute to tunnel degradation potential and pathologies development. Concerning disorders that may be caused by the high weathering potential of calcareous rock masses and evaporites, about 80% of the tunnels detected with this analysis, do indeed show symptoms related to this type of rock mass deterioration. While more than 67% of tunnels that cross rock masses with potential delayed behaviour show long term pathologies that may be related to an increase of pressures on the lining (e.g. convergences, longitudinal cracks and invert heave up). For the others (i.e respectively, 20% and 33% of tunnels) the information got from the last principal inspection is too general to make any kind of particular statement. In this analysis, attention is focused on the disorder type and not on its quantity.

As most of the G.I.S. selected tunnels really show expected symptoms during their service life, this analysis shows that the geotechnical vector map can be used for a first rough identification of tunnels that could develop long term pathologies due to their initial conditions (i.e. geographical location and geotechnical context). In Figure 5.7 are represented all the tunnels of the TDB that may be considered in a potential critical position:

- 37 tunnels (i.e. about 22% of all the tunnels in the TDB) cross rock masses with a potential delayed behaviour,
- 46 tunnels (i.e. about 27% of the total population of the TDB) cross zones with a high weathering potential.

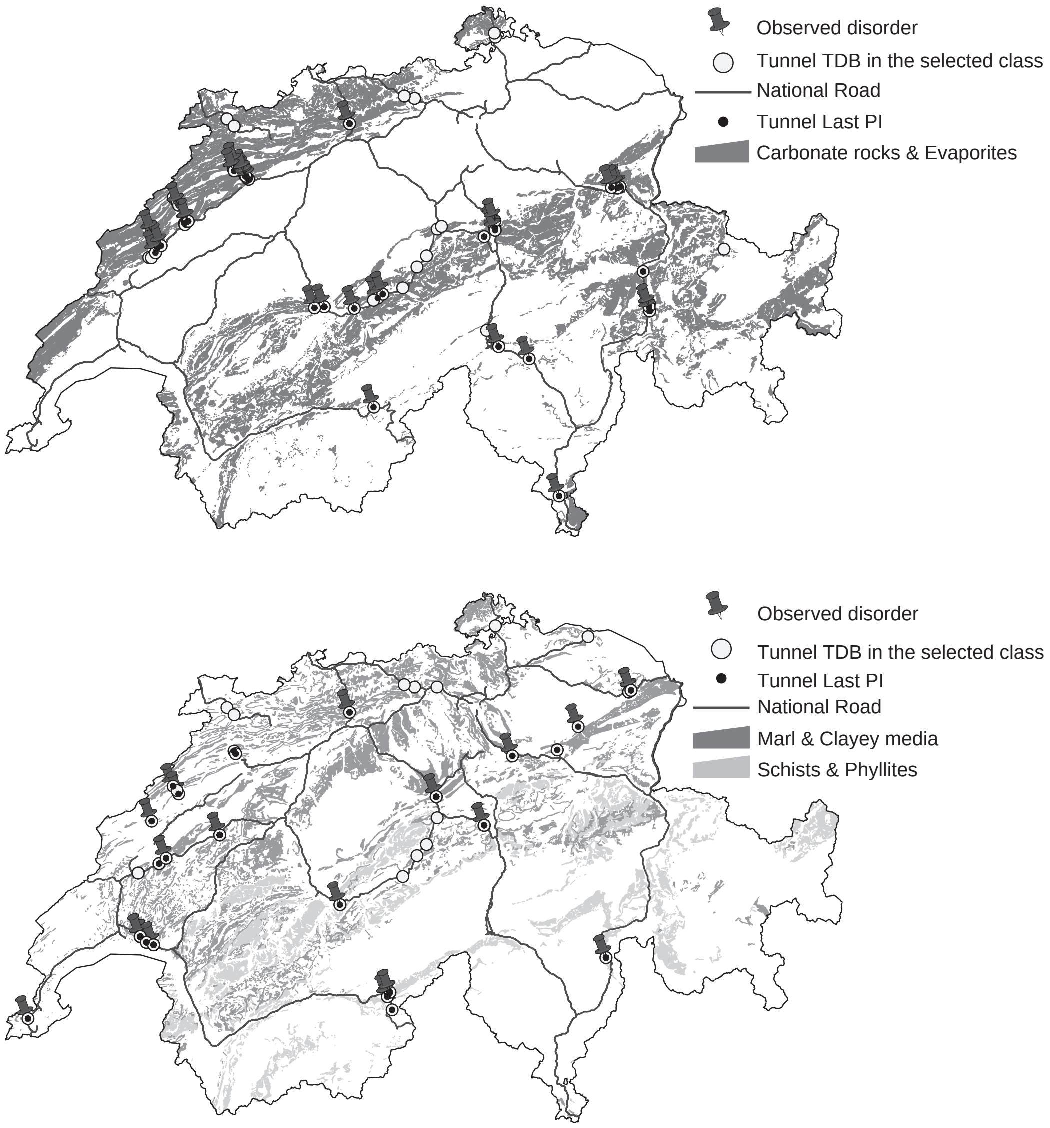


Fig. 5.7. G.I.S. preliminary detection of disorders related to geological conditions (source: Swiss 1:200´000 geotechnical map - OFEV) (On top: National Roads tunnels potentially and really affected by disorders due to rock mass weathering potential. On bottom: National Roads tunnels potentially and really affected by disorders related to rock mass delayed behaviour, (after [127]).

The same procedure has been used considering the hydrogeological conditions (source map: *Esquisse hydrogéologique de la Suisse* - OFEV). Indeed, as already described in Chapter 2, several tunnel disorders depend on ground water circulation around the excavated tunnel. Moreover, the mean number of disorders reported in Figure 5.6, shows how the contribution on tunnel degradation potential of porous rock masses may reveal quite important. As shown in Figure 5.8, the 5 aquifers proposed by the ATLAS Hydrologique (source: Swisstopo, 2000) may be associated to the respective geotechnical classes as follows:

1. Fissured (e.g. in crystalline rocks),
2. Fissured partly porous (e.g. in schists and phyllites),
3. Porous partly fissured (e.g. in carbonate rocks and sandstone),
4. Porous (e.g. in marls and clayey media),
5. Karstified (e.g. in carbonate karst rocks and karst evaporites).

Together with intrinsic permeability and water circulation around the excavation, water pressure on tunnel lining is an important parameter that should be taken into consideration for analysing tunnel long term stability conditions. As already explained in Section 5.1, water circulation around excavation may influence the choice of tunnel waterproofing system. Moreover, the tunnel long term behaviour in terms of lining bearing capacity vs. external pressure depends on drainage and waterproofing system [58]. From statistical analyses results it is possible to identify two categories of problems:

- Tunnels excavated in discontinuous (i.e. fissured, karstified and fissured partly porous) formations may show in the long term water incomes, concrete lining weathering and, in case of low temperatures, ice formation. Moreover, calcareous concretions can appear in the drainage system of tunnels excavated in carbonate karst rock masses (Figure 5.8, top).
- Tunnels excavated in porous rock masses (i.e. porous, porous partly fissured and fissured partly porous) may be affected by pathologies related with delayed behaviour as, for example, invert heave, pressure increase on the tunnel lining due to, for example, drainage filling by fines transport (Figure 5.8, bottom).

As it is possible to see the Figure 5.8 shows for the selected classes the tunnels of the TDB in critical position (white circles), the tunnels for which, there’s available information, in the data base, about disorders recorded during last principal inspection, PI (black points), and the tunnels that really show the expected disorders, (black pins). Also in this analysis only qualitative information is taken into account for each structure. The checked disorders depend on the physical and chemical actions of water (i.e. ***[1]***, ***[2]***, ***[3]***, ***[12]***, ***[17]***) in case of discontinuous and karstified rock masses, while for porous and partly porous (i.e. both fissured partly porous and porous partly fissured) rock masses disorders deal with water contribution to fines transport and swelling potential (i.e. ***[11]***, ***[15]***). Though cracks and fissures can be considered as a consequence of rock mass swelling behaviour as this disorder is quite common to all tunnels excavated in rock masses with delayed behaviour, it has not been considered in this analysis for avoiding any misjudgement. Furthermore, also if tunnel deformations both in crown ***[9]*** and walls ***[10]*** can be a symptom of swelling, too, the number of observed disorder is too small to be taken into consideration.

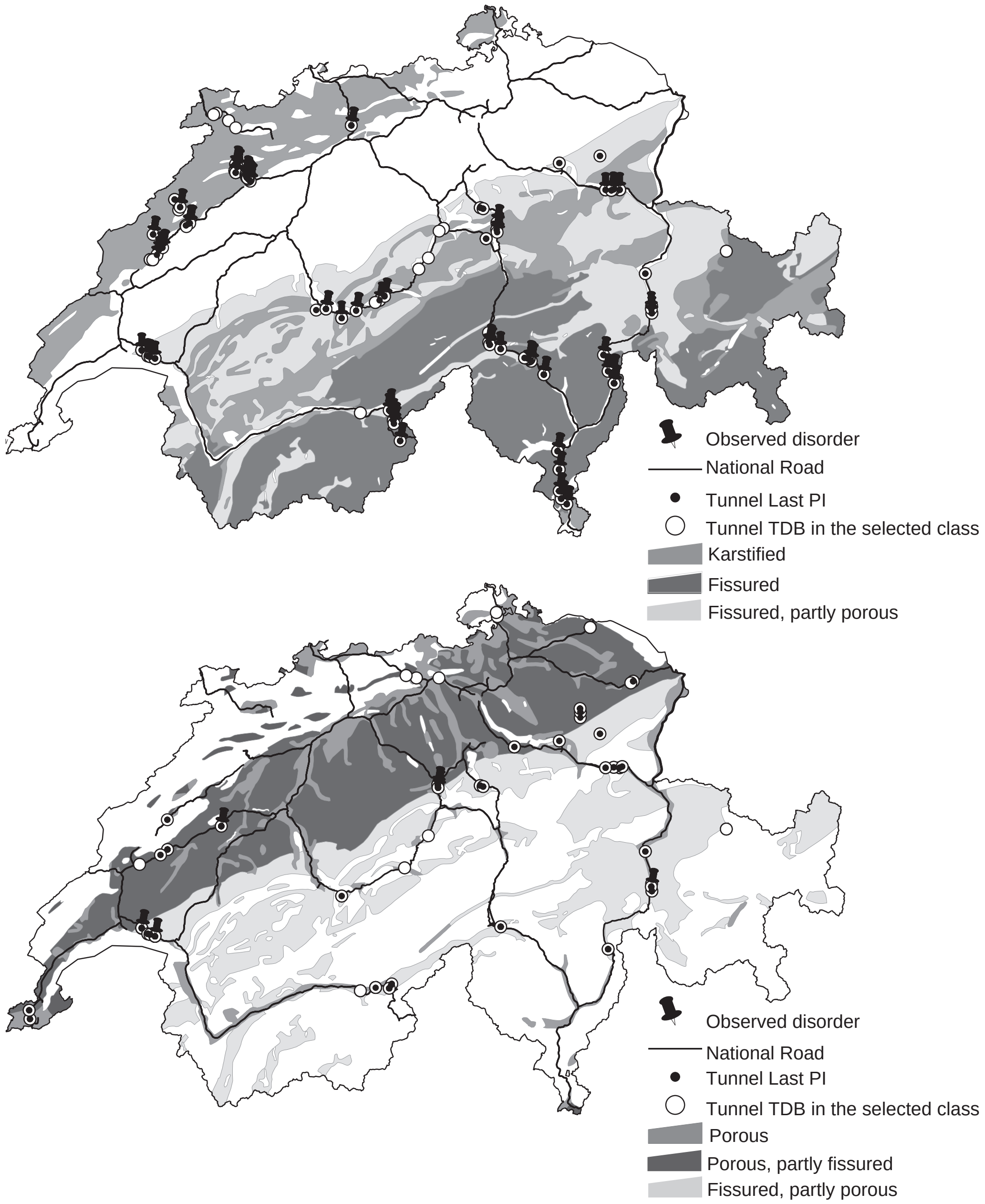


Fig. 5.8. G.I.S. preliminary detection of disorders due to water circulation around excavated tunnels. (Source: Swiss 1:200´000 hydrological map - OFEV). On top: tunnels potentially and really affected by disorders caused by water circulation in discontinuous and karstified rock masses; on bottom: tunnels potentially and really affected by disorders caused by water circulation in porous rock masses.

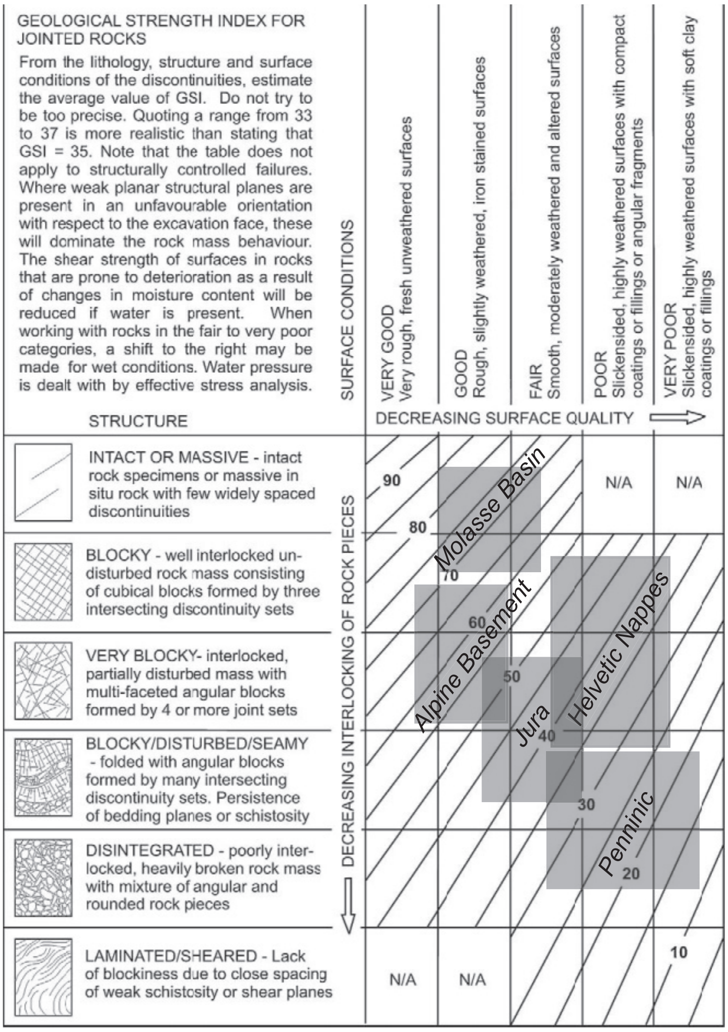
As already explained in Chapter 2, geological conditions, together with tunnel depth influence the tunnel squeezing potential. By superposing the Digital Terrain Model (source: MNT25 - Swisstopo) and the Swiss Geotechnical map (source: Swiss 1:200´000 geotechnical map - OFEV), it should be possible to detect tunnels that match conditions for showing a squeezing behaviour and to check them using information recorded in the data base. To perform this analysis, several data are required. The maximum depth of each tunnel and rock mass unit weight values, γ_{rm}, allow estimating the field stress conditions σ_0 (i.e. the worse in terms of overburden). Moreover, for each geotechnical class, it is possible to evaluate the rock mass strength, σ_{cm}, from the uniaxial compressive strength of the intact rock, σ_{ci}, and Geological Strength Index GSI. Then, as suggested in

[69] (see Chapter 2, Equation 2.23), the squeezing potential is evaluated by calculating the potential strain ε on the base of the competency factor $\frac{\sigma_{cm}}{\sigma_0}$ and the internal support pressure. To take into account the uncertainties of these data, several values were considered:

- mean value and standard deviation for σ_{ci},
- mean, maximum and minimum values for GSI (respectively, $GSI, GSI_{max}, GSI_{min}$)
- 0, 100, 150 and 200 kPa for the internal support pressure.

While the uniaxial compressive strength and the unit weight were estimated from laboratory tests conducted on several rock mass specimens at the Rock Mechanics Laboratory (LMR-EPFL) during past years, *GSI* values have been chosen using the table proposed by Hoek, (2003) [20]. Figure 5.9 shows the chosen values based on the following considerations (see [144] for a detailed geological description of Switzerland). Apart from quaternary deposits:

1. The rock formations of the Jura Mountains (*Jura*) are characterised by folding and thrust, they can be fissured but the quality of the surfaces is quite fair, $GSI = 35 \div 50$. The compressive strength of the intact rock may change from very low values for marls and clays ($\sigma_{ci} \sim 10$ MPa) to high values typical of carbonate rocks ($\sigma_{ci} \sim 80$ MPa).
2. The Molasse Basin (*Molasse Basin*) consists of marls and sandstones sequences, with a intact compressive strength from 2 to 20 MPa, together with very resistant conglomerates ($\sigma_{ci} \sim 100$ MPa). All these formations are mainly undisturbed (massive or blocky), with good conditions of fissures surfaces, $GSI = 65 \div 80$.
3. The formations of the Alps (*Helvetic Nappes*) are very blocky and disturbed rock masses with highly weathered joint surfaces, $GSI = 30 \div 50$. The intact compressive strength of this zone varies from quite high values for carbonate rocks ($\sigma_{ci} > 70$ MPa) to low values for marls ($\sigma_{ci} < 25$ MPa).
4. The Penninic formations (*Penninic*) are very disturbed and schistose rock masses characterised by moderate values of intact compressive strength ($\sigma_{ci} \sim 40$ MPa), with highly weathered joint surfaces and possible clay fillings, $GSI = 15 \div 35$.
5. The Alpine Basement formations (*Alpine Basement*) are characterised by a high intact compressive strength ($\sigma_{ci} > 80$ MPa) but they can be blocky and very blocky formations with an important number of joints sets, in good conditions and with rough surfaces, $GSI = 50 \div 65$.



From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis.

Fig. 5.9. Geological Strength Index (GSI) values (after Hoek, (2003) in [20]) for main Swiss geological formations (after [144]). The labels correspond to the formations described in the text (see text in brackets).

The squeezing potential of the national Roads tunnels recorded in the TDB has been first estimated for the medium set of mechanical parameters for unsupported tunnels. Then, to investigate the influence of these parameters on the squeezing potential of the rock mass, the presence of a support has been considered and the geological parameters were varied within the range reported in Figure 5.10.

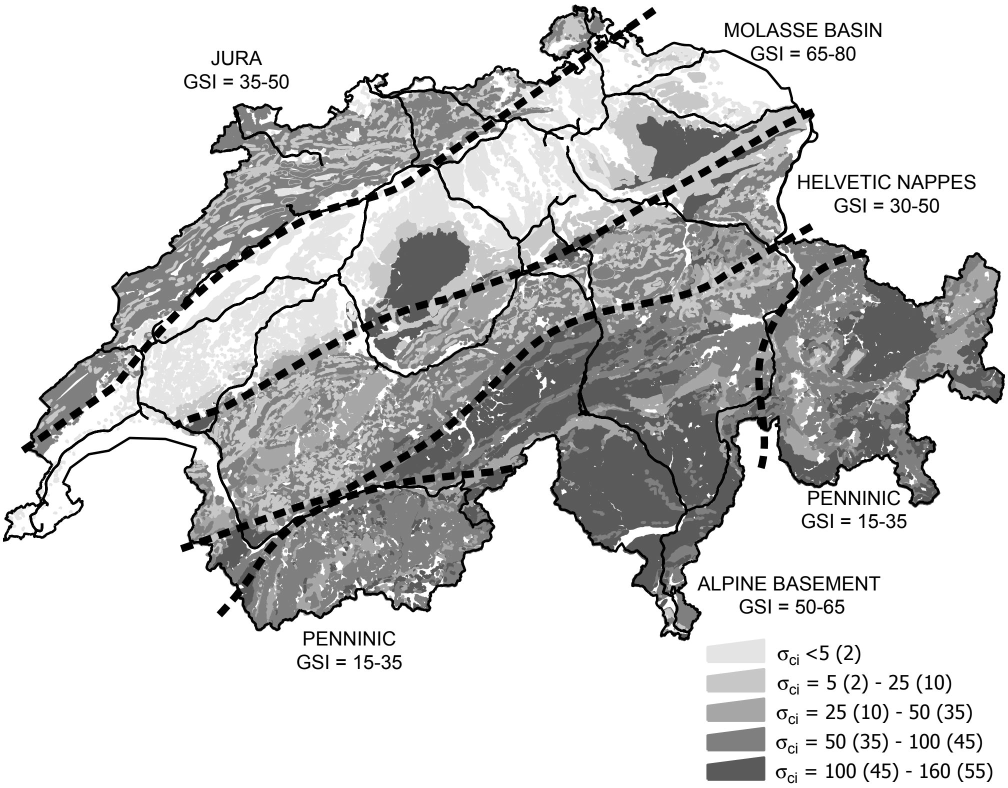


Fig. 5.10. Geological formation characterisation for squeezing potential analysis: mean value and standard deviation of uniaxial compressive strength of intact rock ($\sigma_{ci}(STD)$*), together with* $GSI_{min} \div GSI_{max}$ *for each rock formation described above. As the analyisis is done by using the geotechnical classes of the vector map (source: OFEV), each point of the map is characterised by a precise value and this figure represents only a simplification.*

As shown in Table 5.3, mainly due to its small values compared with the overburden pressure it seems that the internal support pressure has a rather small influence on the final result. While, GSI index and intact compressive strength may change in a considerable way the squeezing potential of the rock mass around the tunnel. Low values of GSI (i.e. GSI_{min}) as well as low intact compressive strengths (i.e. $\sigma_{ci} - STD$, confidence interval: 68%; $\sigma_{ci} - 2 \cdot STD$, confidence interval: 95%) increase the number of tunnels with very high squeezing potential of the rock mass.

Tab. 5.3. From top: influence of the internal support pressure, the uniaxial compressive strength of the intact rock and GSI on the squeezing potential around Swiss National Roads tunnels (VH = very high; H = high; M = moderate; L = low). The reference case is reported in gray filled rows.

			number of tunnels with squeezing potential			
σ_{ci}	GSI	**pressure (kPa)**	**VH**	**H**	**M**	**L**
σ_{ci}	GSI	0	7	5	0	5
		100	3	3	6	1
		150	2	2	7	1
		200	2	2	2	5
$\sigma_{ci} - (2 \cdot STD)$			32	9	8	5
$\sigma_{ci} - (STD)$			12	3	9	9
σ_{ci}	GSI	0	7	5	0	5
$\sigma_{ci} + (STD)$			2	1	3	3
$\sigma_{ci} + (2 \cdot STD)$			0	2	1	5
	GSI_{min}		14	2	5	9
σ_{ci}	GSI	0	7	5	0	5
	GSI_{max}		0	5	5	2

Then, by means of G.I.S. tools, the squeezing potential results have been checked by using the Tunnel Data Base information about fissures and deformations observed during principal inspections. Less than 42% of tunnels with high and very high squeezing potential (considering the results from the analysis with mean values of both intact compressive strength and GSI, with an internal pressure, p_i equal to 0 kPa) really shows the expected disorders. Moreover, excluding cracks and fissures that may depend on several influence factors, very few cases of crown and walls deformation have been recorded. If a lining is well designed, it may take a long period for a tunnel before showing the expected disorders. As a matter of fact, in most cases, the inspected tunnels do not show manifest symptoms of squeezing behaviour. Thus, instead of checking the observed disorders, it could be more relevant to analyse constructive details as section shape, support and lining type and thickness. For example, as shown in Table 5.4, in the majority of tunnels identified with a high and very high squeezing potential, difficulties arose during excavation. Moreover, the tunnel shape is circular or horseshoe shaped with an invert (I), the support thickness is quite important, with steel sets (e.g. HEB240) or intensive rock bolting (II), together with a definitive lining thicker than usual (i.e. 40-45 cm vs. 30 cm) and/ or reinforced concrete is used at least at the tunnel invert (III).

Tab. 5.4. Typical construction features for tunnels with high and very high squeezing potential. Features labels, described previously, are detailed in the table footnotes.

Tunnel ID (TDB)	I[a]	II[b]	III[c]
18	X	X	X
19	X	X	X
28	X	(X)	X
29	X	(X)	X
106		X	X
141	X	(X)	X
167	X	X	
168	X	X	
190		X	X
207	X		X
208	X		X
301		X	

[a] I = Circular section / Horseshoe shaped section with invert

[b] II = Steel sets (e.g. HEB240) / Intensive rock bolting / (>20 cm)

[c] III = Lining thicker than normal (e.g. >30 cm) / Reinforced concrete

5.2.2 Correspondence Analysis: Influence factor modality/attribute vs. Disorder type

After establishing tunnel initial conditions and degradation potential by means of simple statistical methods, it is necessary to explore data for analysing the influence of each factor on the tunnel degradation rate. As already written in Chapter 2 and demonstrated in the previous paragraph by means of G.I.S. analyses, each disorder may depend on one or more influence factors.

In Figure 5.11 for each type of disorder it is shown the number of tunnels affected as it results after the last tunnels principal inspection. There is no information about the presence of voids behind the tunnel lining ***[7]*** neither about local walls deformation ***[10]***. Also local crown deformation ***[9]*** may be neglected. Fissures ***[8]***, water incomes ***[1]***, reinforced concrete corrosion ***[6]*** are most widespread disorders. When a disorder appears on several tunnels, it may reveal quite complicated to find particular conditions of pathology development, however, it may be possible, by using appropriate analytic processes (e.g. Correspondence Analysis, already mentioned in Section 5.1), to find relationships that link influence factors modalities/attributes and observed disorders. These analysis process allows describing the most relevant dependencies between disorders and influence factors.

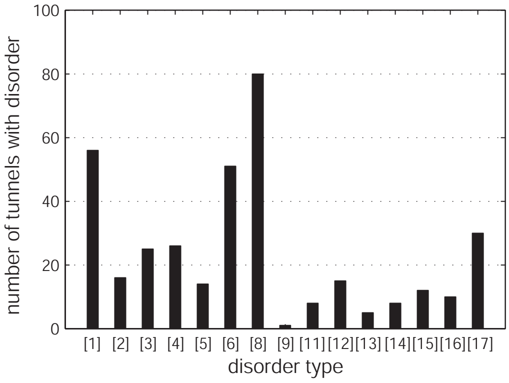


Fig. 5.11. Number of tunnels affected by each type of disorder. Only last principal inspection data have been considered. Each disorder has been associated to a number (for labels meanings see Table 4.7, Chapter 4).

Following the same procedure as in the previous paragraph for evaluating tunnel degradation potential, factors that are supposed influencing degradation rate have been selected within all the features that characterise the tunnel during its service life. As already introduced in Chapter 2, tunnel degradation rate is a consequence of service life conditions and duration. Among the factors considered for evaluating their influence on the tunnel degradation rate it is possible to identify some of the influence factors that have already been considered in the first part of the analysis (i.e. as initial conditions). This is due to the fact that the excavated rock mass is a part of the tunnel and evolves with time by changing its global conditions. Together with the age of the tunnel, for characterising road tunnels service life conditions it is necessary to consider:

1. operation conditions,
2. climatic conditions,
3. geological and hydrogeological conditions (i.e. ground water aggressive action).

In terms of collected information, this corresponds to focus attention on traffic volume, ventilation system, temperature and humidity (from which it depends the quality of the atmosphere inside the tunnel) and also on waterproofing system and chemical composition of ground water. As reported already in Chapter 4, some of this data comes from alternative information sources (e.g. Federal Offices) others than Cantonal responsibles.

For identifying relationships between selected features and pathologies by means of Correspondence Analyses, data are resumed in several contingency tables of $n_{rows} \times m_{columns}$. The number of columns ($m_{columns}$) represents the problem dimension which corresponds to the number of attributes of the potential influence factors, while the number of rows (n_{rows}) represents the disorders. As better explained in [60]; [92]; Nishisato, (2004) (see [126]), Correspondence Analysis transforms a complex multidimensional problem in a simpler 2-dimensional scatter plot, called bi-plot. The most interesting feature of this bi-plot is that it allows to represent together influence factor modalities (columns pattern) and disorders (row points). The distance between row points represents how disorders are similar to each others with regard to the influence factor modality. Moreover, the distance between column-points and row-points describes the eventual dependency of the disorder from the influence factor attribute and allows to identify recurrent pathologies. Thus, by interpreting Correspondence Analysis results it is possible to identify factors that determine tunnel degradation rate. Analyses results for the selected features are shown in figures from Figure 5.12 to Figure 5.17. In order to avoid data redundancy, also in this case, only disorders recorded during the last principal inspection have been considered.

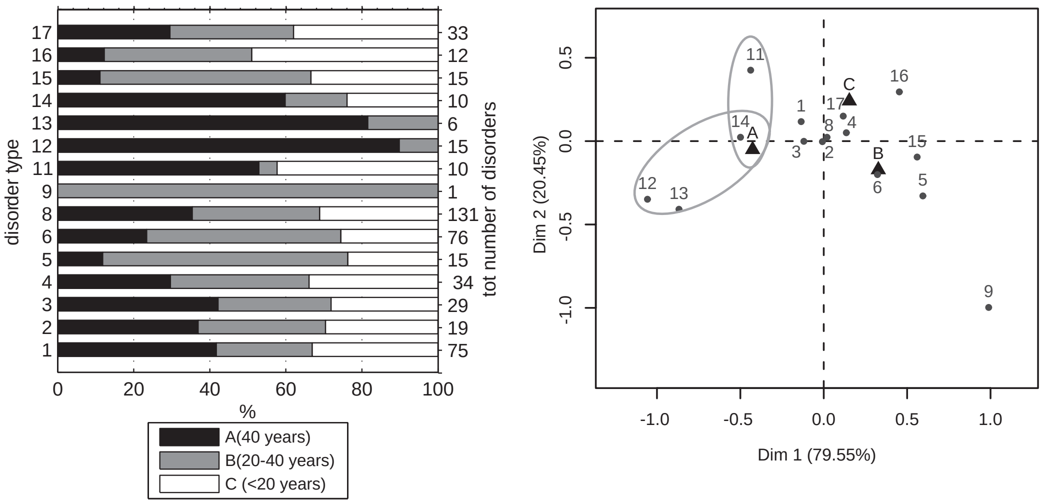


Fig. 5.12. Correspondence Analysis results: tunnel disorders vs. age of the tunnel. Tunnel age is calculated starting from the commissioning year; while disorders are detailed in Table 4.7, Chapter 4.

SERVICE LIFE DURATION, OPERATION & CLIMATIC CONDITIONS. Considering that age plays an important role for all kind of structures, first analyses have focused on the influence of tunnel age on its degradation speed/rate (see Figure 5.12). Tunnel age represents the service life duration, thus it is calculated starting from the commissioning year. Due to the ageing process of construction/building materials, 40 years old structures as several Swiss National Roads tunnels may show disorders on concrete linings, ***[13]***. Also track conditions ***[14]*** may depends on service life length. Furthermore, some of the old tunnels were constructed without waterproofing system and this may explain the fact that disorders due to ice formation ***[12]*** characterise this class of structures. The poor quality of lining material and casting conditions, together with the absence of waterproofing system may justify fines transport in the drainage system ***[11]***. Finally, due to their position in the bi-plot, water incomes ***[1]***, efflorescence ***[2]***, concrete degradation ***[3]***, and cracks ***[8]*** cannot be associated to any of the age classes in particular.

Service life duration influence should be considered also in terms of operation conditions. As a matter of fact, traffic volume plays an important role in pathologies development during tunnel service life. As already shown in Chapter 2, traffic is considered an important cause of impact damages and track scaling. While the impact damages ***[16]*** depend clearly on very high traffic conditions, on the contrary of expectations, track scaling ***[14]*** characterises tunnels with medium traffic conditions. This can be justified with the fact that when the traffic is high and very high the track maintenance is more frequent and this reduce the number of observed disorders during principal inspections. Furthermore, due to the projections of water together with de-icing salts particles, traffic may contribute to the gutter and side walls corrosion (i.e. up to 1.5 m high) ***[4]***, ***[5]*** and ***[6]***. Finally, the corrosive action of exhaust gases, together with humidity, can contribute to efflorescence formation ***[2]*** on the concrete of the lining. Figure 5.13 on top shows how using Correspondence Analysis it has been found that the above-mentioned disorders characterise tunnels with high and very high traffic conditions. At the same time, it is possible to observe that also tunnels with low traffic are affected by de-icing salts corrosive action (i.e. disorders ***[4]***, ***[5]*** and ***[6]***). This can be ascribed to the fact that a significant number of low traffic tunnels are very old and/or are located at a high altitude. Actually, both quantity of de-icing salts (applications frequency depends on weather conditions as it is explained in the following) and concrete quality (which mainly depends on the age of the structure) contribute to lining deterioration by de-icing salts.

Moreover, together with traffic also the ventilation system contributes to the atmosphere quality inside the tunnel. In particular, as demonstrated at the beginning of this chapter, old and short tunnels are generally naturally ventilated. This may contribute to lining deterioration due to humidity and pollution inside the tunnel. On the bottom of Figure 5.13 it is shown that tunnels with natural ventilation are affected by concrete deterioration, ***[3]*** and ***[13]*** and efflorescence ***[2]***, due to corrosive atmosphere inside the tunnel. Though in the case of natural ventilation lining deterioration may be ascribed also to the age of the structures, as the majority of tunnels without ventilation system has been constructed before the Nineties, the graphic shows how the same disorders affect tunnels equipped with longitudinal ventilation system. In this case, the age of the structures has a lower influence as this type of ventilation is used also in recent tunnels. Furthermore, it is interesting to observe that tunnels with transversal and semi-transversal ventilation system seem to not be affected by disorders caused by traffic pollution. It is thus possible to affirm that also in case of recent tunnels (i.e. good lining materials and casting quality) if the pollution is not directly evacuated from the tunnel the concrete lining may suffer of pathologies caused by aggressive atmosphere.

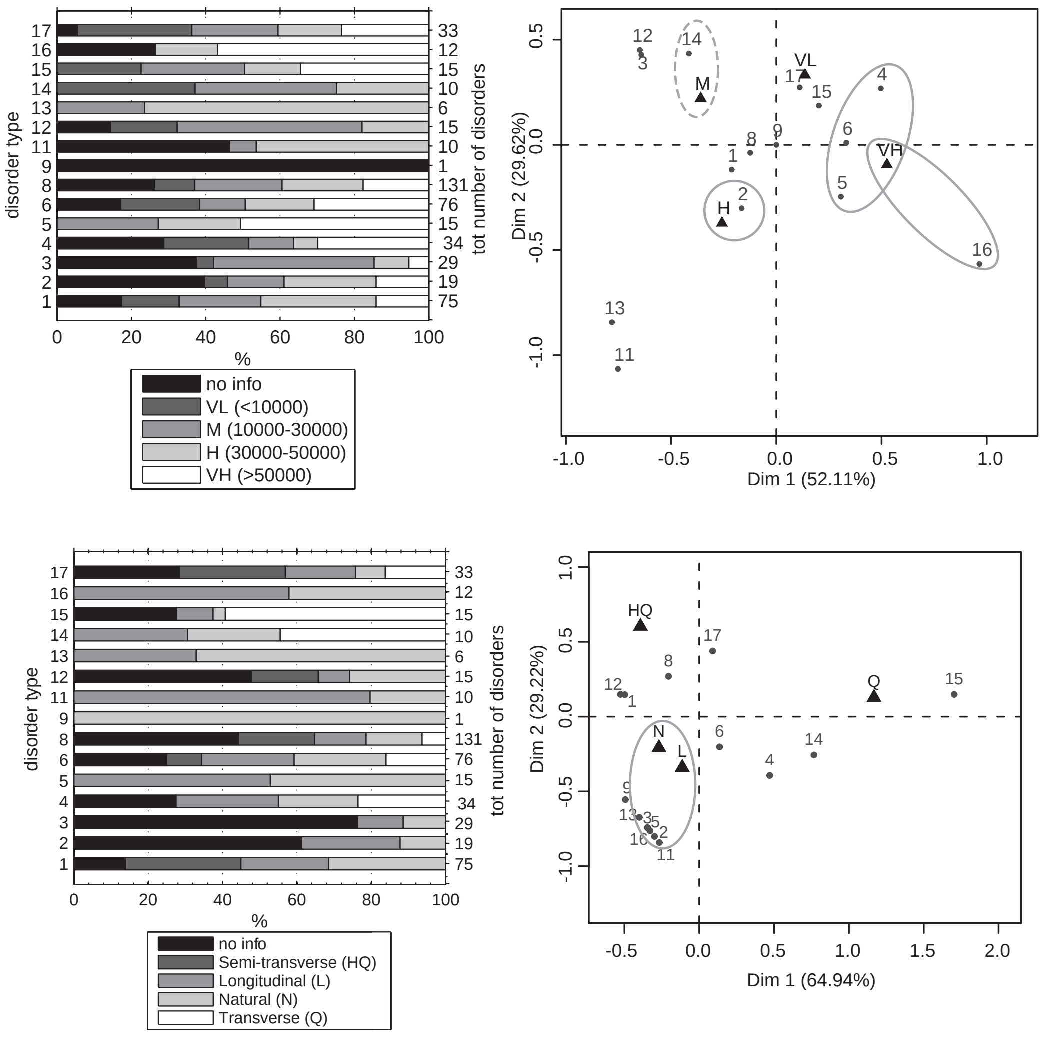


Fig. 5.13. Correspondence Analysis results. On top: tunnel disorders vs. average daily traffic volume, DTV (data source: OFROU, 2006). On bottom: tunnel disorders vs. tunnel ventilation system. The disorders have been detailed in Table 4.7, Chapter 4.

Using these results, together with vectorised values of average daily traffic volume (DTV), it is possible to detect tunnels in potential critical conditions by using G.I.S. tools. Figure 5.14 shows how the 90% of tunnels with high and very high daily traffic volumes show at least one of the disorders identified with the Correspondence Analysis (see Figure 5.13). Thus, by extending detection to all tunnels in the data base, as it was done with previous G.I.S. analyses, it has been found that almost the 30% of all the TDB population may potentially develop disorders caused by high traffic conditions. Furthermore, considering principal inspection information collected, for each disorder, it has been evaluated the percentage of events that depends on high/very high traffic conditions, neglecting tunnels without information about traffic conditions:

- Efflorescence ***[2]***: more than 54%;
- De-icing salts corrosion for plain and reinforced concrete ***[4]***, ***[5]*** and ***[6]***: more than 60%;
- Track scaling ***[14]***: about 25%;
- Damages due to car collisions ***[16]***: 100%.

It is significant that damages due to car collisions appear almost only in tunnels with high and very high traffic conditions, while track scaling seems to depend more on the length of the tunnel service life (as shown in Figure 5.12). Anyway, for what concerns track scaling, as observed previously, a more frequent track maintenance may reduce the number of observed disorders. Moreover, it is interesting to point out how important is the contribution of traffic to side walls corrosion due to de-icing salts, while concrete efflorescence is more related with the aggressive action of water on the lining as it will be shown further below (Figure 5.18).

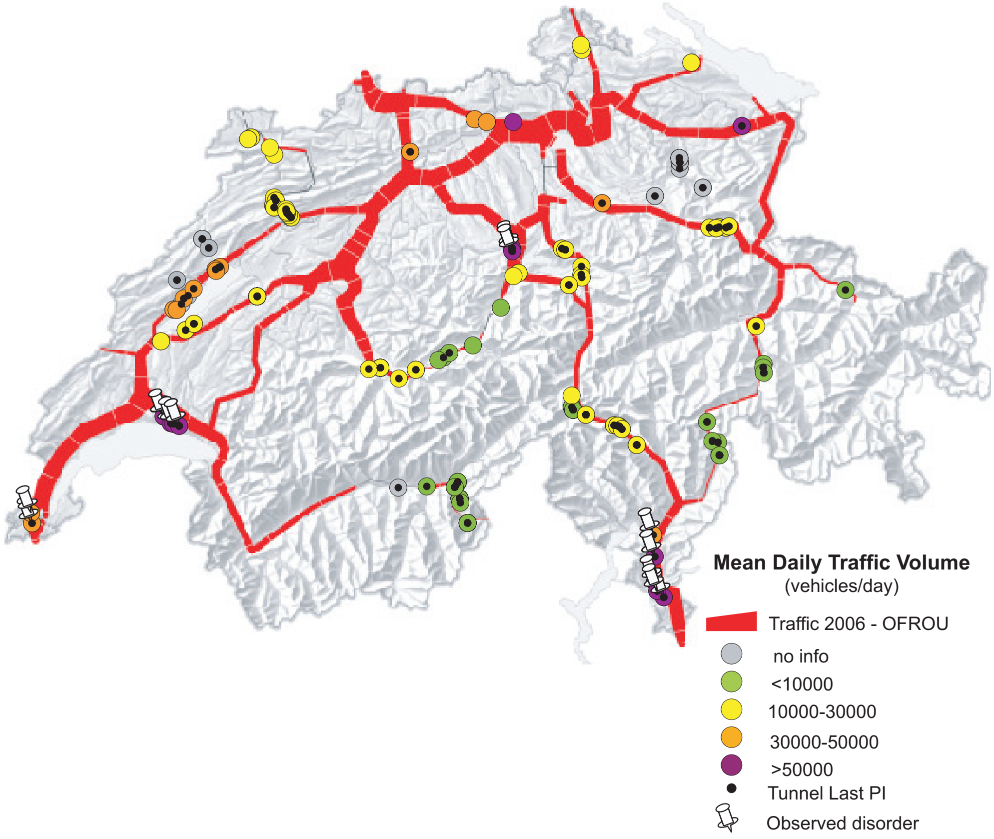


Fig. 5.14. G.I.S. preliminary detection of tunnels with disorders due to traffic circulation (source: OFROU, 2006). Tunnels potentially (i.e. orange and purple circles = high traffic expressed in mean number of vehicles per day) and really affected by disorders as efflorescence due to traffic pollution, de-icing salts corrosion, track scaling and car collisions damages (i.e. checked disorders respectively: ***[2]***, ***[4]***, ***[5]***, ***[6]***, ***[14]***, ***[16]****).*

Though the importance of service life length is quite clear, above presented analyses have been performed by considering only values (i.e. average daily traffic volume, DTV) relative to the year 2006. For evaluating the degradation rate due to operation conditions it is necessary to estimate the total traffic volume during the whole tunnel service life. More detailed information, provided by OFROU, about the average daily traffic volume on Swiss National Roads between 1985 and 2006, allows evaluating traffic evolution during the last 20 years. Information covers the main National Roads axes and local values are given for several measurement points. Within this "traffic data base", using G.I.S. tools, it is possible to get the traffic information for the tunnels of the National Roads network. The traffic evolution data show that traffic grows regularly with time, but the evolution rate is faster for roads with high and very high average daily traffic volume. Thus, though those data cover only 20 years, under the condition that traffic grows regularly with time, by extrapolation and successive integration, it has been possible to estimate for each tunnel the traffic charge during whole service life (i.e. from commissioning year to present). Four main classes have been distinguished:

- Low traffic charge: < 300'000 vehicles,
- Medium traffic charge: 300'000 - 600'000 vehicles,
- High traffic charge: 600'000 - 1'200'000 vehicles,
- Very High traffic charge: > 1'200'000 vehicles.

This new analysis by containing in itself two different informations (i.e. service life length and traffic volume) and allows evaluating the contribution of both influence factors on tunnel degradation rate. Figure 5.15 shows Correspondence Analysis results. It is interesting to observe that with expected disorders as impact damages ***[16]*** and de-icing salts aggression ***[5]*** and ***[6]*** in this case also concrete degradation ***[13]*** depends on very high traffic charges. This can be explained by considering that very high traffic charges depends on traffic conditions during the whole tunnel service life and may represent old tunnels with low quality of lining material operating in aggressive environment.

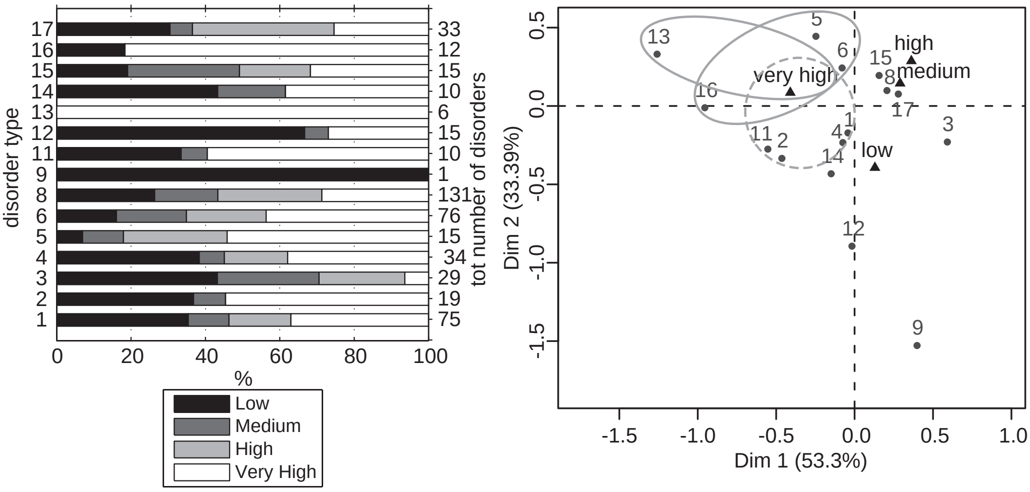


Fig. 5.15. Correspondence Analysis results: tunnel disorders vs. total traffic volume. Disorders list has been presented in Table 4.7, Chapter 4.

Finally, in order to refine evaluation of traffic contribution to disorders development, and, thus, distinguish between mechanical (e.g. car collisions) and chemical (e.g. corrosion) actions, de-icing salts quantity has been considered. As prescribed by the VSS norm [147] (art. 9), when external temperature is less than 0°C, de-icing salts use is compulsory for roads with unfavourable humidity conditions. Though in reality several factors contribute to ice formation on the roads [46], this information allows estimating the contribution of de-icing salts to the development of related tunnel pathologies, by means of G.I.S. analysis, only considering temperature information (i.e. weather and altitude data).

Using measurements from several weather stations by MétéoSuisse, [33] in his PhD Thesis identified in Switzerland six distinct regions and estimated, for each region, the average de-icing salts quantity used during winter season. According to his results it is possible to affirm that the Alps, together with the Jura Mountains and the Engadine region are characterised by the worse conditions, followed by the Leman region and the Swiss "Plateau". For the southern part of Ticino canton, due to the mild weather, he estimated the smallest quantity of de-icing salts. Figure 5.16 shows the results obtained by means of G.I.S. detection based on these considerations. By checking data from last tunnel principal inspections, within the detected tunnels, more than 61% really show the expected disorders (i.e. concrete damage and reinforced concrete corrosion by de-icing salts). Moreover, the 35% is represented by tunnels located in the Alpine region. However, by analysing these results, some important remarks should be done:

- Though, according to [33], the southern part of Ticino canton is characterised by mild weather and thus low quantities of de-icing salts, by considering the tunnels altitude, together with information collected directly from the cantonal responsible, the low quantity of de-icing salts appears underestimated and should be upgraded to a medium quantity (i.e. at least the same that has been considered for the Leman region).
- In spite of the high quantity of de-icing salt estimated for tunnels located in the Jura mountains, the number of tunnels showing effectively disorders due to de-icing salts attack is quite low. In this case it is important to consider that, due to its seasonal character this kind of pathology probably depends on the tunnel age. Thus, recent tunnels may not yet show corrosion disorders.
- High traffic volumes may accentuate the de-icing salts corrosion as it happens for tunnels located in the Leman region and in the central part of the Swiss "Plateau".

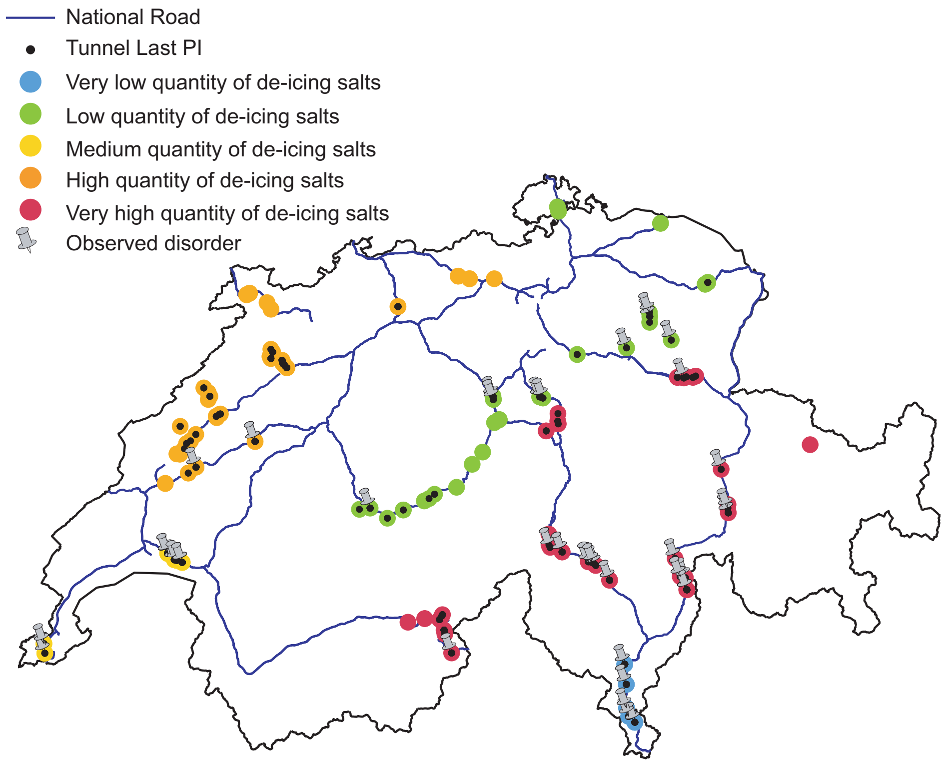


Fig. 5.16. De-icing salts contribution to walls corrosion by Chlorides. Quantity values have been estimated mainly based on climatic conditions (after [33]).

Geological & Hydrogeological conditions. Together with operation conditions, as introduced in Chapter 2, geological and hydrogeological conditions may have an important contribution on the tunnel degradation rate. The graph on top of Figure 5.17 shows how evaporites influence the development of calcareous concretions in the tunnel drainage system ***[17]*** together with efflorescence ***[2]*** and lining instabilities ***[13]***. Water incomes ***[1]*** and ice formations ***[12]*** characterise tunnels excavated in calcareous rock masses probably due to the presence of discontinuities in this kind of formation. For the same reason, water incomes ***[1]*** and concrete degradation ***[3]*** (e.g. calcium leaching) are more common in tunnels that cross discontinuous and blocky rock masses (e.g. gneiss). Tunnels excavated in marl and clayey media and loose ground are characterised by fines transport in the drainage system ***[11]***. For what concerns invert heave ***[15]***, due to its position, any particular influence can be identified. As it is possible to see in the resulting bi-plot, also the disorder number ***[8]*** is placed in the middle of the pattern described by the different geological formations. Mainly due to the fact that many factors contribute to lining cracking (see Chapter 2), this disorder can be hardly associated to a precise geological class.

As shown previously, for the evaluation of the tunnel degradation potential due to initial conditions, by considering together geological conditions and depth it is possible to evaluate the squeezing potential of the rock mass. Thus, in terms of tunnel degradation rate, depth can contribute to the development of disorders induced by the delayed behaviour of rock masses. Correspondence Analysis results, on bottom of Figure 5.17, show how tunnel invert deformation ***[15]*** is related to deep tunnels. Also track scaling ***[14]*** seems to be caused by the high depth of the tunnels. This may be explained by considering that, when invert deformation happens in tunnels without invert, it is possible to identify track disorders as a manifest symptom of invert heave. Crack and fissures ***[8]*** characterise tunnel with low and medium overburden, probably due to the bending moment caused by asymmetric loading conditions, more accentuated at a low depth. Water incomes ***[1]*** and other disorders related to water infiltration are mainly observed in shallow tunnels (i.e. ***[2]***, ***[3]***, ***[11]***, ***[12]***, ***[13]***), perhaps due to the fact that, together with ground water, also surface water has an easy way to the tunnel. As a matter of fact, water incomes are quite common also for tunnel portals, where the depth is very low and the tunnel crosses, very often, loose ground masses (i.e. in particular ***[11]***).

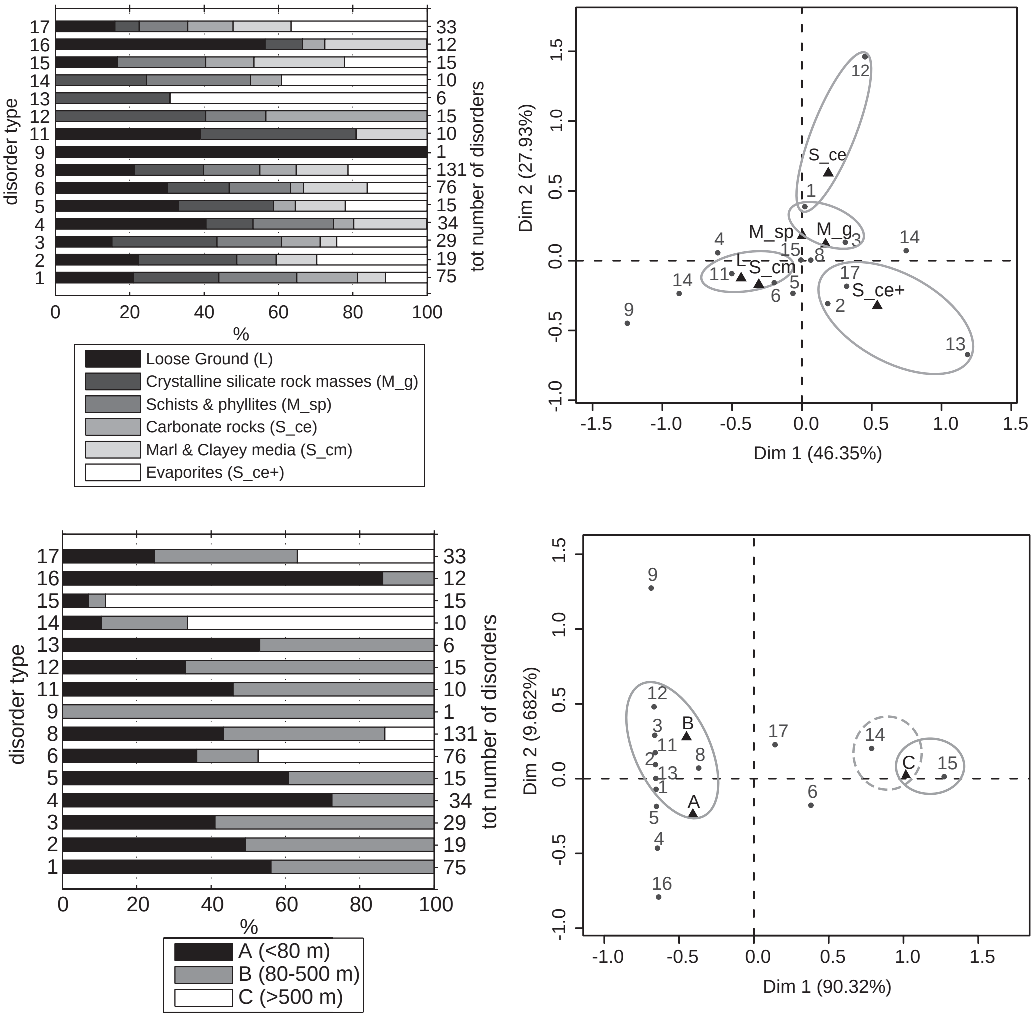


Fig. 5.17. Correspondence Analysis results. On top: tunnel disorders vs. tunnel geological conditions. On bottom: tunnel disorders vs. tunnel depth. Disorders list has been shown in Table 4.7, Chapter 4.

Some results of Figure 5.17 top are influenced by considerations about tunnel hydrogeological conditions, as lining weathering may strongly depend on ground water chemical composition. A literature review has been performed for better interpreting the hydrogeological influence on disorders development. [150] in his book shows how the chemical composition of ground water may affect tunnel conditions both on the short and on the long term. As already demonstrated by G.I.S detection (see Section 5.2.1) chemical weathering (alteration) is most active where water percolates through the voids of the rock either intermittently or continuously. As reported by several authors [66]; [76]; [118]; [150], chemical characteristics of circulating water around excavation are mainly controlled by rock mass chemistry (e.g. gneissic, sedimentary carbonate and/or evaporitic rocks) and the extent of water-rock reaction (i.e. tunnel depth and presence of water around excavation). Also the geothermal systems, under particular conditions of ground water temperature and mineral content (e.g. Yverdon les Bains or Schinznach Bad in Aargau Canton), may reveal particularly aggressive for the concrete lining. Anyway, in normal conditions, water temperature is usually close to the expected values for a regional geothermal gradient. Thus for the following analysis only mineral content of rock will be considered for determining the ground water aggressive action.

The ATLAS Hydrologique, (source: Swisstopo, 2000) identifies the six major aquifers in Switzerland (Table 5.5).

Tab. 5.5. Six major aquifers in Switzerland (source: ATLAS Hydrologique, Swisstopo, 2000).

Type	Rock Mass
1	Fluvial/Alluvial Sediments (recent Quaternary)
2	Fluvio-Glacial Deposits (old Quaternary)
3	Molasse rocks
4	Carbonate karst rocks
5	Karstic Evaporites (Gypsum and Anhydrite)
6	Crystalline silicate rocks

In their PhD theses, [66] and [76] show that the chemical composition of ground water in Quaternary Sediments (i.e. type 1 and 2, Table 5.5) and in the Sedimentary formation of Molasse (Helvetica Plate) is practically the same.

[150] divides Switzerland into three main zones based on the chemical composition of ground water:

1. Jura (Northern Switzerland),
2. Mittelland (Central Switzerland),
3. Alps (Southern Switzerland).

Further information comes from [16]. For characterising the durability of waterproofing membranes, he identifies 4 classes of ground water according to the chemical composition and its potential aggressive action against structures.

As introduced in Chapter 2, the presence of sulphates can be very dangerous for concrete structures. As reported by [118], the two main SO_4 sources in Switzerland are:

- Water leaching through upper Triassic sulphate minerals (i.e. Evaporites of Jura mountains and Swiss Rhone Basin).
- Oxidating dissolution of sulphide minerals of crystalline rocks (i.e. pyrite oxidation reaction, Equation 5.1) in the Gotthard area, and in general throughout the Alpine Basement:

$$FeS_2 + \frac{15}{4}O_2 + \frac{7}{2}H_2O \Rightarrow Fe(OH)_3 + 2 \cdot SO_4^{2-} + 4 \cdot H^+ \quad (5.1)$$

Finally, according to [142], it is possible to characterise the aggressive action of ground water by considering the water acidity: in general, a pH of about 6.5-6.7 is already considered quite aggressive for concrete structures. [66]; [76] state that in general, ground water is characterised by a neutral or alkaline pH (up to more than 8 in case of high carbonates contents) while acid pH can be measured in water from siliceous rocks.

In the framework of this research, a new classification is proposed based on the above mentioned considerations and data (Table 5.6). For each type of rock formation it is reported the most probable ground water chemical composition together with the corresponding geographical location in Switzerland.

Tab. 5.6. Rock type, ground water pH, major chemical components and location in Switzerland. Values in brackets are less common in the respective zones.

Rock	**GW pH**	**GW composition**	**Location in Switzerland**
Crystalline Gneiss	5.9-6.8	Na^+, Ca^{2+}, (Mg^{2+}), HCO_3^-, (SO_4^-)	Alpine Basement
Carbonate rocks	7.2	Ca^{2+}, HCO_3^-, (Mg^{2+})	Jura Mountains, Helvetic nappes
Evaporites	7.1-7.5	SO_4^-, Ca^{2+}, Mg^{2+}, HCO_3^-	Pre-Alps (Swiss Rhône basin) and Jura Mountains
Porous and fissured molasse, sandstone, conglomerates and quaternary deposits	6.5-8.1	HCO_3^-, Ca^{2+}, Mg^{2+}, Cl^-, (SO_4^-), (Na^+)	Molasse Basin
Flysch and schists	5.9-6.8	Ca^{2+}, HCO_3^-, (Mg^{2+})	Penninic

By using the above presented information about chemical composition and position of the aquifer, together with both the hydrological vector map (source: OFEV) describing water circulation type, and the geotechnical vector map (source: OFEV) describing the rocks formations, it is possible to identify, through G.I.S. analysis, where tunnels are supposed to show symptoms of ground water attack during their service life. For example, calcareous concretions in the drainage system ***[17]*** are mainly expected in carbonates and evaporites. While concrete degradation due to calcium leaching ***[3]*** and efflorescence ***[2]*** may characterise not only carbonates and evaporites but also crystalline formations. Moreover, in the worse cases, a prolonged sulphate corrosive action may cause lining crumbling ***[13]***. Anyway, water coming from evaporites have a higher sulphate content than in crystalline rock masses, thus lining pathologies due to concrete corrosion by sulphates attack are expected more in the Jura Mountains than in the Alps.

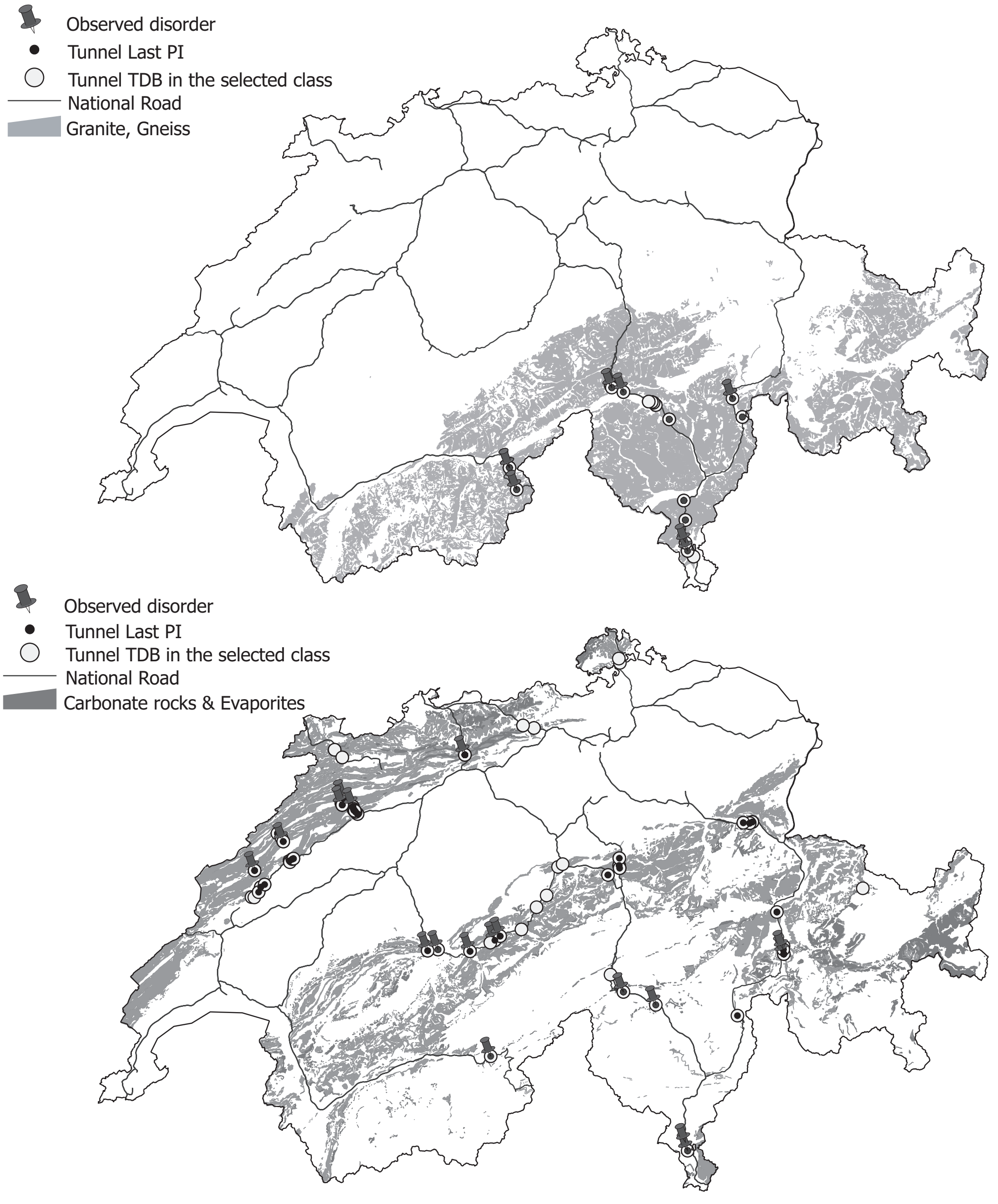


*Fig. 5.18. G.I.S. preliminary detection of disorders due to ground water chemical composition. (Sources: Swiss 1:200´000 geotechnical map & hydrological map - OFEV). On top: tunnels crossing crystalline formations and thus potentially and really affected by disorders caused by sulphates **[2]**, **[13]** and concrete deterioration **[3]**. On bottom: tunnels crossing carbonates and evaporites formations and thus potentially and really affected by disorders caused by sulphates **[2]**, **[13]** concrete deterioration **[3]** and calcareous concretion in the drainage system **[17]**.*

Figure 5.18 shows G.I.S. analysis results. For each ground water type, potential disorders have been selected within road tunnels disorders list (Table 4.7, Chapter 4), in particular:

- concrete degradation by calcium leaching ***[3]*** and sulphates ***[2]***, ***[13]*** together with calcareous concretion ***[17]***, for ground water in evaporites and carbonates formations;
- concrete degradation by calcium leaching ***[3]*** and sulphates ***[2]***, for ground water characterising crystalline rocks.

Then, using information collected in the TDB about disorders observed during the last principal inspection, it has been checked whether tunnels identified in the G.I.S. analysis are really affected by the selected pathologies. Among tunnels excavated in calcareous rock masses, about the 43% shows calcareous concretions in the drainage system ***[17]*** and concrete lining weathering due to water leaching ***[3]***. While, 66% of tunnels excavated in gneiss and crystalline rocks shows water incomes and other disorders related to water chemical composition.

However, as already mentioned, it is necessary to keep in mind that ground water action may be reduced depending on type and position of the waterproofing system. As expected, (Figure 5.19 on top) the absence of waterproofing, causes the development of all those disorders that depend on ground water flow into the tunnel: efflorescence ***[2]***, concrete lining weathering ***[3]*** and ***[13]***, and ice formation ***[12]***. Probably due to the fact that water incomes ***[1]*** depend not only on the waterproofing system, but also on the concrete lining conditions, the analysis results do not allow to associate them to a particular waterproofing system. Further explication may need also the fact that concrete concretions in the drainage system ***[17]*** seem to depend from the absence of waterproofing. Water circulation around the tunnel lining in absence of waterproofing system allows ground water to leak into the concrete and charge in calcium which can, on the long term, precipitate in the drainage system. This hypothesis is supported also by the fact, mentioned previously, that the absence of waterproofing characterises mainly old tunnels. It is rather surprising to observe that tunnels with an intrados waterproofing system are characterised by pathologies that depend on relative humidity of atmosphere inside the tunnel as, for example, the corrosive action of de-icing salts. This phenomenon may be explained by the fact that this kind of waterproofing is normally placed only on tunnel crown, while the above mentioned pathologies affect mainly the gutter and the first 1.5 m of side walls (see Figure 2.7). As a matter of fact, the internal waterproofing system works as an umbrella, water percolating through the lining, moves along the extrados of the side panels, reaches and saturates the lower part of the side walls. Moreover, as explained by [2] this system has a shorter life compared to the membranes placed between temporary support and definitive lining or at the tunnel extrados. Though invert heave up seems to depend on waterproofing partial system, this result should be interpreted carefully. Indeed, as invert heave mainly occurs in tunnels with horseshoe shaped section and those tunnels have generally a partial waterproofing system (Pm) between the support and the lining, there is a bias in the Correspondence Analysis.

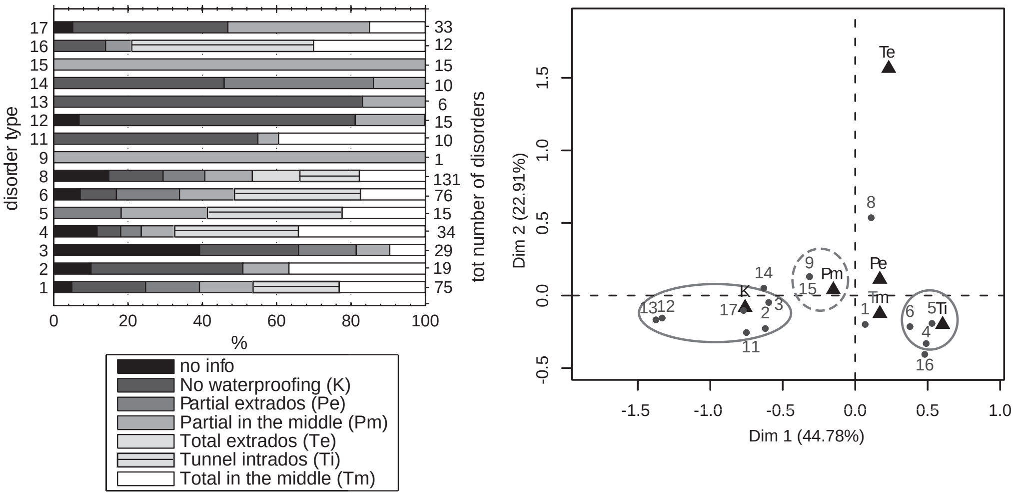


Fig. 5.19. Correspondence Analysis results. Tunnel disorders vs. tunnel waterproofing system. The list of disorders is detailed in Table 4.7, Chapter 4.

5.2.3 Influence factors selection

By means of the Correspondence Analyses described in the previous paragraph, main influence factors that determine tunnel degradation rate have been pointed out. Table 5.7 shows for each influence factor the significant (expected and/or unexpected) dependencies after interpretation and analysis of above presented results. Due to the small number of observations (see Figure 5.11) disorder 9 won’t be further considered. Moreover, as it is possible to observe in Table 5.7, three disorders are marked by a question mark (‘?’). Indeed, the Correspondence Analysis results did not highlight any particular dependency as in the three cases the relative disorder was found in the middle of the influence factors pattern. Anyway, mainly due to considerations from the literature review (Chapter 2), together with degradation potential analyses (Section 5.2.1), it has been decided to verify whether preliminary expectations could have been confirmed by this last step of the analysis.

Tab. 5.7. Synthesis of the factors influencing the tunnel degradation rate: for each influence factor it is indicated the list of dependent disorders as it results after interpretation of Correspondence Analyses results (see Paragraph 5.2.2). Factors analysed also by means of G.I.S. tools are written in italics. The disorders list is provided in Table 4.7, Chapter 4.

Influence Factor	**1**	**2**	**3**	**4**	**5**	**6**	**8**	**11**	**12**	**13**	**14**	**15**	**16**	**17**
Age - Service Life length								X	X	X	X			
Traffic (DTV - 2006)		X		X	X	X							X	
Ventilation system		X	X		X					X				
Total traffic volume		X		X	X	X				X			X	
Geology / Hydrogeology	X	X	X					X	X	X		?		X
Tunnel depth	X	X	X				X	X	X	X		X		
Ground water chemical composition		X	X							X				X
Waterproofing system	?	X	X	X	X	X		X	X	X				X
Tunnel shape							?					X		

For each influence factor there are several attributes/modalities. As shown already with Correspondence Analyses, only some of them really contribute to the development of identified disorders. To highlight the relationships between some factor modalities and observed disorders, further analysis have been performed. To achieve this goal, the procedure was as follows:

1. For each disorder selection of the tunnels of the TDB affected by this disorder (according to the last principal inspection results);
2. For each influence factor, distribution of tunnel population into modality classes (the number of classes corresponds to the number of attributes of each factor).
3. Evaluation of the probability that a tunnel belonging to a certain modality class shows actually the corresponding disorder.

For each factor modality class, the probability, (which is a conditional probability), is calculated as follows:

$$P(disorder\,/\,factor\ modality) = \left(\frac{\text{number of affected tunnels}}{\text{tot. number of tunnels}}\right)_{\text{factor modality class}} \tag{5.2}$$

The probability values per influence factor, expressed in percent, are summarised in Table 5.8. Since the selected influence factors are current tunnel features, this table allows to determine the attributes/modalities that mainly contribute to each disorder occurrence and to identify the most favourable scenario for its development. When the probability value is larger than 40%, the influence is considered as significant (values in *italics* in Table 5.8).

Tab. 5.8. Disorder probability in percent (note that the character "-" stands for 0%). The disorders list is provided in Table 4.7, Chapter 4; while, for each influence factor the attributes have been described in Section 5.2.2. The results for the three disorders marked by a question mark in Table 5.7, are reported in gray filled rows

Influence Factor	**Results**					
Age	A	B	C			
11	17	2	7			
12	41	5	-			
13	14	2	-			
14	17	5	7			
Traffic (DTV)	Low	Medium	High	Very High		
2	6	8	18	14		
4	38	12	9	50		
5	-	12	14	36		
6	69	22	50	86		
16	-	-	9	43		
Ventilation system	L	N	Q	HQ		
2	18	10	-	-		
3	20	20	-	-		
5	16	12	-	-		
13	4	6	-	-		
Geology / Hydrogeology	L	M_g	M_sp	S_cm	S_ce	S_ce+
1	*40*	*64*	36	42	46	40
2	20	32	14	20	-	40
3	20	32	29	17	5	40
11	40	21	-	20	-	-
12	-	18	7	-	19	-
13	-	16	-	-	-	20
15	-	-	14	24	10	20
17	10	14	21	42	27	*40*
Depth	<80	80-500	>500			
1	51	42	-			
2	15	12	-			
3	24	16	-			
8	71	58	33			
11	5	9	-			
12	15	7	-			
13	4	5	-			
15	9	7	67			

Influence Factor	**Results**				
Waterproofing system	K	Pe	Pm	Ti	Tm
1	86	50	46	100	54
2	43	-	13	-	23
3	29	25	15	-	8
4	14	13	29	75	46
5	-	13	28	25	15
6	29	75	54	100	31
11	43	-	4	-	15
12	57	-	15	-	-
13	29	-	4	-	-
17	43	-	35	-	15
Shape	H	HR	K		
8	67	65	56		
15	11	10	6		

Since the results are similar, daily traffic conditions and total traffic volume are considered together. The same for geological and hydrogeological conditions as they both depend on rock formation type.

By comparing the results, it is possible to summarise the influence of each factor considering disorders one by one:

[1] Water incomes and moisture are quite widespread, they are mainly influenced by waterproofing system, geological and hydrogeological conditions and tunnel depth. As the probability of observing water incomes is quite high with all kind of waterproofing, by means of the Correspondence Analysis any particular dependency was observed. Anyway, waterproofing system is an important factor for observing water incomes in the tunnel. In particular, the probability of having water incomes is higher in case of tunnels without sealing (K) and/or equipped with an internal waterproofing system (Ti). Another factor that influences the probability of water incomes is the geological conditions, the probability is higher in discontinuous rock masses. Finally, the probability increase in case of low depth tunnels.

[2] Efflorescence mainly depends on ground water action. High probability values characterise tunnels without waterproofing system as well as tunnels that cross evaporites (i.e. high sulphates content) and acid gneissic formations. It is important to notice that the absence of efflorescence in tunnels with internal waterproofing system is probably due to the fact that the waterproofing panels avoid the possibility of observing this kind of disorder which is mainly concentrated in the tunnel crown. Traffic pollution can be considered only as a secondary influence factor: probability values slightly increase in case of high traffic volumes. Moreover, only longitudinal and natural ventilation systems seem to have a small contribution to efflorescence development. A specific control should be performed for verifying the lining conditions above the ventilation slab.

[3] Weathered surfaces due to calcium leaching, staining and calcareous concretions mainly depend on hydrogeological conditions. Water circulating in evaporites and crystalline rocks may influence a lot the development of concrete lining deterioration. Also low tunnel depth and absence of waterproofing system seem to influence the development of this disorder. Finally, it is interesting to observe that this disorder characterise only tunnels with natural or longitudinal ventilation systems. Anyway, probability does not reach remarkable values. Moreover, this can also be due to the fact that in case of transverse and semi-transverse ventilation systems, the tunnel crown is not visible. These results confirm the expectation that the aggressive

geological and hydrogeological environment can be considered as the main cause of this disorder.

[4] Concrete spalling, delaminated concrete, together with disorders no. ***[5]*** (plain concrete damage by de-icing salts) and ***[6]*** (reinforced concrete corrosion by Chlorides) depend on traffic conditions. The probability of these disorders increases with increasing traffic volumes. However, a high probability of reinforced concrete corrosion ***[6]*** is found also in case of low traffic volumes. This can be explained by the fact that the main cause of this disorder remains the quantity of de-icing salts projected on tunnel lining during its service life, as shown also by considering the total traffic volumes results. Indeed, as explained in Section 5.2.2, high quantities of salt are used also for low traffic tunnels. This happens, for example, for tunnels located at a high altitude and thus subjected to harsh weather conditions, with severe temperature decrease. Moreover, those tunnels are probably less maintained than tunnels with high daily traffic volume. A particular remark should be done about the influence of the waterproofing system. As mentioned in Chapter 2, these disorders depend on the concrete lining saturation which may increase in case of internal waterproofing system. Anyway, the high values that characterise also tunnels with partial waterproofing placed between the support and the lining or at the support extrados confirm that traffic has a major influence on the development of these disorders.

[5] see disorder ***[4]***

[6] see disorder ***[4]***

[7] Based on present TDB information, this type of disorder has not been recorded during last tunnels principal inspections (see Figure 5.11).

[8] Cracks and fissures depend on tunnel depth and also shape seems to have slight influence. As already suggested by [40], the probability of having cracks, due to the above ground mass mechanical properties decrease and confinement loss, is higher in shallow tunnels. The loss of axisymmetric loading conditions induces a bending moment in the tunnel lining. When the tensile stresses are too high, a fissure (usually longitudinal) may appear in the structure. The results confirm the difficulty in identifying cracks origins without a detailed characterisation of type, aspect, position, length and width. However, probability slightly increases for horseshoe shaped tunnels as circular sections normally better support external loads.

[9] Due to the small number of observations reported in last tunnels principal inspections (i.e. only one record), local deformations of crown are not considered in this analysis (see Figure 5.11).

[10] Based on present TDB information, this type of disorder has not been recorded during last tunnels principal inspections (see Figure 5.11).

[11] Fines transport in the drainage system depends on tunnel geological conditions: this disorder mainly occurs in tunnels excavated in loose ground masses and clay formations. Moreover, also the absence of waterproofing system may influence its development. As waterproofing depends on tunnel age it is interesting to see that a slight increase of this disorder is recorded, as expected, in case of old tunnels. The poor quality of old lining materials, together with casting conditions may allow the fines leakage.

[12] The probability of observing ice formation is high in tunnels without waterproofing system. As this kind of tunnels were usually constructed before the seventies this explain also the high values that characterise very old tunnels. On the contrary, this disorder is not observed in tunnel with total waterproofing. Though less important, also depth and hydrogeological conditions influence the development of this disorder, higher probability values characterise tunnels excavated in fissured and blocky rock masses (i.e. gneiss and carbonates) and/or at a low depth, where the ground water may find an easier way to the tunnel.

[13] As this disorder has been observed only in few cases (i.e. 6 records), the probabilities of occurrence are very low. Anyway, some considerations can be done. Local lining failures and concrete blocks falls seem to depend on the waterproofing system as the highest probability value characterises tunnels without waterproofing system. The importance of the age of the tunnel can be observed also by observing that this disorder affects only tunnels with a very high total traffic volume that characterises, among others, long duration of service life. Though the probability values are too low for identifying any

dependency, the aggressive atmosphere inside the tunnel can be considered also as an influence factor as this disorder is observed only in tunnels with natural and longitudinal ventilations. Considering tunnel geological conditions, blocky rock masses can cause lining instabilities due to unexpected local loads to bear. An higher probability value characterise as well evaporites probably due to the swelling pressure that this kind of rock mass may develop with time in case of chemical reaction with water (see Chapter 2). Moreover, also the sulphates content of this kind of rock mass may cause lining degradation till crumbling.

[14] Track scaling has a high probability to affect old tunnels, because track damage increases with time. Against any expectations, the traffic seems not to influence this disorder, but this can be explained by the fact that in case of high and very high traffic volumes the track maintenance is done more frequently than in tunnels with low traffic conditions. Thus, this result shows the importance of the conservation procedures for reducing tunnel degradation rate.

[15] Invert heave up is a typical symptom of rock mass delayed behaviour: the probability is very high in case of very deep tunnels, and this is due to the high confinement conditions. For what concerns geological conditions probability increases in case of clays and marls and evaporites. Finally, the shape of the tunnel contributes to the development of this disorder: a slight increase of probability values is observed for horseshoe shaped tunnels. As a matter of fact, a circular section, with a closed lining, better supports asymmetric loading pressures.

[16] Impact damages only depend on tunnel operation conditions as a high probability characterises only tunnels with very high traffic volumes.

[17] The drainage system conditions are strongly influenced by geological conditions: the probability of observing drainage system obstructions by calcareous concretions is higher in sedimentary rocks as carbonates and evaporites. This kind of disorder is more probable in tunnels without waterproofing system probably due to the poor quality of the concrete lining (old tunnels, as mentioned above) and to the fact that ground water may increase its calcium content when in contact with the concrete lining. Moreover, a high probability value characterises also tunnels with partial waterproofing at the lining extrados. This may be due to the water leaching through the shotcrete support, a slow degradation process that characterises tunnels since the beginning of their service life.

In many cases, a certain disorder is influenced by several factors; moreover, a disorder may develop due to a combination of well defined factors modalities. Thus, for identifying and describing main pathologies scenarios it is necessary to analyse also the combination of influence factors modalities. By definition, this corresponds to evaluate the joint probability which describes the probability of two events in conjunction. If the two events are independent the probability of both events together is expressed as the simple product of their individual probabilities. While, if the occurrence of one event does affect the probability of the other occurring, then the events are dependent and the probability of one event occurring if another event has already occurred is called conditional probability. In this case, joint probability has been evaluated under two main simplifying conditions:

1. only the highest probability values have been considered,
2. in case of interaction between more than two factors the interaction has been considered first between the highest values 2 by 2 only, then it has been evaluated also the eventual contribution of a third or a fourth factor.

After calculation of the joint probability between independent factors, Table 5.9 summarises the main scenarios for development of each disorder. It is important to observe that the conditional probability has not been calculated because, as observed at the beginning of this chapter, in this case, the population of each group would have be too small to be considered as representative.

Tab. 5.9. Main scenarios for disorder development: for each disorder, it is indicated whether it depends on one or more influence factors modalities. For each factor, the modalities are listed according to their importance as estimated with probability values. For independent factors, the interaction has been evaluated by calculating the joint probability. Significant interaction (+++) corresponds to a joint probability >40%; moderate interaction (++) corresponds to a joint probability between 30% and 40%, (+) slight interaction stands for values between 15% and 30%.

Disorder	**Scenario**	**Interaction**
[1]	$\text{Waterproofing}_{Ti,\,K}$, $\text{Geology}_{M\_g,\,S\_ce}$, $\text{Depth}_{low,\,medium}$	
	$\sum(\text{Waterproofing}_{Ti,\,K} + \text{Geology}_{M\_g,\,S\_ce})$	+++ to ++
	$\sum(\text{Waterproofing}_{Ti,\,K} + \text{Depth}_{low,\,medium})$	+++ to ++
	$\sum(\text{Geology}_{M\_g,\,S\_ce} + \text{Depth}_{low,\,medium})$	++ to +
	$\sum(\text{Waterproofing}_{Ti,\,K} + \text{Geology}_{M\_g,\,S\_ce} + \text{Depth}_{low,\,medium})$	++ to +
[2]	Waterproofing_{K}, $\text{Geology}_{S\_ce+,\,M\_g}$	
	$\sum(\text{Waterproofing}_{K} + \text{Geology}_{S\_ce+})$	+
[3]	Geology_{S_ce+}, Waterproofing_{K} [a]	
[4]	$\text{Waterproofing}_{Ti}$, Traffic_{VH}	
	$\sum(\text{Waterproofing}_{Ti} + \text{Traffic}_{VH})$	++
[5]	Traffic_{VH}, $\text{Waterproofing}_{Ti}$	
[6]	$\text{Waterproofing}_{Ti}$, $\text{Traffic}_{VH,\,H,\,L}$	
	$\sum(\text{Waterproofing}_{Ti} + \text{Traffic}_{VH,\,H,\,L})$	+++
[8]	$\text{Depth}_{low,\,medium}$, $\text{Shape}_{H,\,HR}$	
	$\sum(\text{Shape}_{H,\,HR} + \text{Depth}_{low,\,medium})$	+++ to ++
[11]	Waterproofing_{K}, Geology_{L}	
	$\sum(\text{Waterproofing}_{K}, \text{Geology}_{L})$	+
[12][b]	Waterproofing_{K}, $\text{Age}_{very\ old}$	
[13][b]	Waterproofing_{K}, $\text{Total traffic volume}_{VH}$, $\text{Geology}_{S\_ce+,\,M\_g}$, $\text{Age}_{very\ old}$	
[14]	$\text{Age}_{very\ old}$	
[15]	$\text{Depth}_{very\ deep}$, $\text{Geology}_{S\_cm,\,S\_ce+}$ [c], $\text{Shape}_{H,\,HR}$ [c]	
	$\sum(\text{Geology}_{S\_cm} + \text{Depth}_{very\ deep})$	+
[16]	Traffic_{VH}	

Disorder	Scenario	Interaction
[17]	$\text{Waterproofing}_{K,\,Pm}$, $\text{Geology}_{S\_\text{cm},\ S\_\text{ce+}}$	
	$\sum(\text{Waterproofing}_{K,\,Pm} + \text{Geology}_{S\_\text{cm},\ S\_\text{ce+}})$	+

a.The absence of waterproofing system is considered though its probability value is less than 40%, as a matter of fact for developing this kind of disorder the aggressive ground water should find a way to the lining.

b.Age and waterproofing system are dependent variables (Section 5.2.1).

c.These factors should be considered only as secondary factors as probability values are quite small compared with the influence of depth.

Some comments can be done by looking at the results of Table 5.9.

Water inflow ***[1]*** depends on waterproofing system, geological conditions and tunnel depth. The interaction of these factors have a strong influence on the development of this disorder, too. For, example, the absence of waterproofing system in tunnels excavated in fissured rock masses can cause a lot of water incomes (e.g. as it happens in the Taubenloch tunnels between Canton of Bern and Canton of Jura or for some tunnels in the Neuchatel Canton). Moreover, when the tunnel is at a low depth, water inflow may be observed more frequently especially during rain falls when the geology allows an easy way for the water to reach the tunnel without waterproofing system.

Efflorescence ***[2]*** and concrete deterioration ***[3]*** are caused by aggressive ground water (i.e. evaporitic rocks) that may leak into the tunnel when there’s no waterproofing system. The absence of waterproofing system may also influence the fines transport in the drainage system ***[11]*** when tunnels are excavated in loose ground masses.

Disorders caused by de-icing salts attack ***[4]***, ***[5]***, ***[6]*** depend on traffic conditions and waterproofing system. The possibility of water to saturate the lower part of the tunnel side walls and the de-icing salts projection by rolling traffic together may reveal particularly aggressive for concrete lining (reinforced or not). Also the duration of tunnel service life is important in the development of these disorders that may affect tunnels with low daily traffic conditions but very high total traffic volumes (i.e. long service life). Furthermore, high traffic volumes are also the cause of impact damages ***[16]***.

Cracks and fissures ***[8]*** are more probable in horseshoe shaped tunnels, especially at low depth. Indeed, as the loads, in this case, may not respect the axisymmetric conditions, bending moments develop in the lining. Compared to horseshoe shaped sections, circular sections better bear such actions, avoiding the development of fissures. Anyway, the results show how also other kinds of section may be affected by this disorder. This is mainly due to the fact that several causes may contribute to fissures development. A detailed description is thus required in case of cracks and fissures for identifying their main causes and operating effective repairs. Moreover, as introduced in Chapter 2, it is important to observe that cracks and fissures may also be the symptom of Alkali-Aggregate Reaction (AAR). Only in recent years this phenomenon has been actually identified in Switzerland on several types of structures, in particular dams and bridges. In 2003, the Federal Roads Authority sent a check list to all responsibles for structures inspections and maintenance describing the main symptoms of this process. At present, it is still quite difficult to recognise the symptoms of this process by means of simple visual inspections. Thus, mainly due to the fact that this pathology may take several years before becoming visible (i.e. more than 20 years, according to [91]), and that Swiss inspectors just begin to be aware about this problem, AAR has not been taken into account in this framework.

When there’s no waterproofing system tunnels may show also other disorders due to water infiltration, as, for example, ice formation ***[12]***. The apparent dependency of this disorder on tunnel age is due to the fact that many tunnels were constructed without waterproofing system. Of course, though they were not taken into consideration in the

Correspondence Analysis due to a lack of information, also climatic conditions may influence the development of this disorder.

Concrete deterioration, local failure and block falls ***[13]*** can be considered as a final stage on the life of the tunnel lining. Though seldom observed, this disorder seems to depend on the aggressive operative and environmental conditions of the tunnel. High total traffic volumes and aggressive water may contribute to its development. The continuous corrosive action of sulphates (evaporitic formations) in absence of waterproofing system, for example, may cause lining weathering till crumbling. Actually, it is important to observe that the absence of waterproofing depends on the tunnel age, and that old tunnels may also be characterised by high total traffic volumes. Moreover, the poor quality of old concrete linings together with higher humidity may contribute to the development of this disorder. The age of the structure may thus be considered an important factor for observing this kind of disorder, especially in absence of regular maintenance.

The age of the tunnel seems to be the main cause of track scaling ***[14]***. Actually, as explained above, this result should be considered with particular caution. As a matter of fact, track scaling probably depends on high traffic conditions, but the more frequent track maintenance that characterises tunnels with high DTV may hide the influence of traffic on the development of this disorder. Again, the importance of regular maintenance is pointed out.

As expected, invert heave ***[15]*** is a symptom of delayed behaviour of the rock mass. Moreover, the probability of this disorder to develop is higher when the tunnel is very deep, while the horseshoe shape of the cross section may be considered only as a secondary influence factor for the development of this disorder.

Finally, calcareous concretion in the drainage system ***[17]*** mostly depend on the chemical content of ground water. Also the absence of waterproofing system and partial waterproofing systems contribute to the development of this disorder. As a matter of fact, the interaction of these two influence factor seem to have a slight influence on the development of this disorder. It is necessary to note that old structures, without waterproofing system, are characterised by a poorer quality of concrete that is more easily affected by water leakage. Moreover, also when a partial waterproofing system is placed at the lining extrados, by leaching through the shotcrete support, percolating ground water may increase its Calcium content before reaching the drainage system. Finally, the age of the tunnel can be considered an important factor also because this disorder can require long periods to develop.

All these information may reveal quite useful both during tunnel design and maintenance planning as they point out not only the main influence factors that cause the development of several disorders observed in road tunnels during principal inspections, but also their possible interaction and role in changing tunnel conditions with time.

5.3 Summary

The diagnosis of actual tunnels conditions, as suggested by guidelines of Road Authorities of several countries (e.g. France and UK, see Chapter 3), is based on the analysis of a well structured and updated data collection. As shown in this chapter, a correct interpretation of collected data needs to consider all information about construction, environmental and operation conditions.

After describing the main features that characterise National Roads tunnels, tunnel degradation potential, due to tunnel initial conditions, and the degradation rate, due to service life and operational conditions, have been analysed. In order to identify systematic pathologies scenarios, typical exploratory data analysis techniques combined with G.I.S. tools have been used. The whole analysis procedure is summarised in Figure 5.5.

1. INITIAL CONDITIONS - DEGRADATION POTENTIAL. As anticipated in Chapter 2, initial conditions do influence tunnel degradation potential and long term behaviour depends on tunnel initial conditions. In this analysis, the tunnel initial conditions have been chosen within the data base features that describe the tunnel conditions at its construction. The influence has been evaluated by considering the mean number of disorders per tunnel feature class. Construction year and techniques as well as waterproofing system have a significant influence on the development of future disorders. This can be related, especially for old tunnels, to the extent of the damaged zone around the excavation. Moreover, within initial conditions, an important role is covered by geological conditions, together with water circulation around the tunnel. Both these factors have been analysed by means of G.I.S. tools. The tunnel degradation strongly depends on the rock mass type: tunnel disorders are correlated with the delayed behaviour, the weathering potential of the rock mass and the water circulation type in the rock mass. Also the squeezing potential has been determined based on both geological conditions and tunnel depth. Finally, it has been demonstrated how the number of tunnels with potential squeezing behaviour strongly depends on the rock mass mechanical properties (i.e. GSI and compressive strength). Moreover, it has been shown that disorders caused by squeezing may be prevented (or at least delayed) by appropriate tunnel design (Table 5.4).

2. SERVICE LIFE - DEGRADATION RATE. The second step of the analysis allows identifying the main factors that influence tunnel degradation rate. By means of Correspondence Analyses it has been possible to determine systematic dependencies between disorder type and influence factor. Tunnel operation conditions and exposure to weathering agents have been considered. High traffic volumes, together with the use of de-icing salts can be particularly aggressive for the concrete lining. Also the chemical composition of ground water, together with the absence of waterproofing system (analysed by means of G.I.S. tools) accelerates lining weathering rate. Geological and hydrogeological conditions as well as tunnel depth may also increase tunnel deformation rate by modifying stresses distribution and, consequently, loads acting on the lining.

DEGRADATION SCENARIOS. Finally, the probability of occurrence of disorders due to influence factors modalities has been evaluated. Main/representative pathology scenarios have been described by pointing out the factors, or the combination of factors, that influence the disorders development (Section 5.2.3).

5.4 Comments & Recommendations

TUNNEL DATA BASE: INFORMATION QUALITY AND DETAIL. The above presented analysis is founded on the tunnel degradation overview presented in Chapter 2, together with information collected in the Tunnel Data Base (Chapter 4). At present, the quality of available data allows only general considerations about pathologies development. Inspection results and disorders observations, together with a detailed characterisation of the tunnel at the local scale are necessary for correctly interpret tunnel degradation phenomena. Considering the existing procedures (KUBA-DB) for the Swiss National Roads, the following observations can be done:

1. As suggested by [100] guidelines, principal inspections should be documented by appropriate reports. At present, the main problem encountered during data collection has been the lack of a standardised way of recording inspection results. As already specified in Section 5.2.1, the mean number of disorders has been evaluated by considering the number of tunnels and not the excavated length (m). This choice was mainly due to the lack of details about disorder location. To improve the actual system it is recommended to document disorders observations by means of sketches showing their position and severity (as it is already done in France by CETu and in Switzerland by Ticino Canton, for example). In this way, through a regular update of the tunnel sketch it is possible to follow the evolution of each disorder.
2. Since data analyses results show the great influence of geological and hydrogeological conditions, together with construction details, on tunnel degradation, an accurate

documentation at the tunnel local scale should be available together with inspection reports. As written previously, though recognised as an important factor for interpreting tunnel degradation by several countries, only France (FR3) [29]; [31] and Portugal (P1) [136] specify the necessity of collecting all this information. Moreover, for some pathologies related with the excavated rock mass it has been shown how interesting information comes directly from construction details (e.g. squeezing potential has been interpreted using information about temporary support, accidents during construction and definitive lining). Thus, in order to facilitate and improve the interpretation of observed disorders, it is recommended to represent geological and hydrogeological data, together with construction details, in the form of a tunnel profile to be superposed to the inspection sketches.

TOOLS: G.I.S. & MULTIVARIATE STATISTICS. The contribution of each tunnel feature to disorders development has been evaluated by means of several tools. Multivariate statistics methods allow to evaluate the disorder dependency on influence factors. The influence of tunnel geographical location on its long term behaviour has been demonstrated, and, at the same time, used for justifying G.I.S. tools analyses. As a matter of fact, tunnel location determines not only its geological and hydrogeological environment, but also climatic and operation conditions (e.g. in particular, traffic volumes and use of de-icing salts). As shown in Section 5.2, the promising agreement of G.I.S. analyses results and observed disorders allows considering G.I.S. as a tool for preliminary detection of tunnels potential pathologies. An interesting feature of using G.I.S. tools is the possibility of managing all the information stored in the Tunnel Data Base for immediate data analyses and major problems identification. Though as recommended by [100] the diagnosis of the tunnel conditions should result from detailed survey activities, a preliminary rough analysis of data may reveal quite interesting for prioritising interventions and/or deciding about preventive maintenance. At present, within the methods suggested by several countries for centralising, storing and processing information about road structures, only Italy (I5) [LaMonica, 2001] proposes the use of G.I.S tools. In order to complete the data collection it is thus suggested to keep the location information (coordinates X,Y) in the Tunnel Data Base (see Paragraph 4.4), using tunnel coordinates (X, Y) for a geographical representation of the available information by means of G.I.S. tools.

PROCEDURE. The TDB analysis procedure used in this work, is inspired from what is already done by OFROU for evaluating National Roads bridges conditions [102]. Though similar to the KUBA-MS procedure (Table 5.10), before becoming a tool for tunnels maintenance decisions, the main purpose of the presented analysis is to identify the influence factors that contribute to tunnel degradation with time. KUBA-MS procedure has a third step for simulating pathologies evolution by means of Markov Chains. In order to perform this analysis, successive inspections results are required. As a matter of fact, the importance of updating the collected data is prescribed by guidelines of several countries (i.e. FR1, CH7, UK1, US1, US2 & SA1, the document codes have been detailed in Chapter 3, Table 3.1 and Table 3.2) and thus recommended. However at present, the Tunnel Data Base does not contain enough, and detailed, information about tunnels successive inspections for estimating time evolution of observed disorders. Actually, since it is not possible to interpret disorders evolution by means of successive observations, degradation models presented in Chapter 2 will be used in the next chapter for predicting/estimating the trend of tunnels long term behaviour by means of analytical formulations.

Tab. 5.10. Comparison between KUBA -MS and TDB analysis procedure: structure and methods.

	KUBA-MS	**TDB analysis**
Road Structure:	Bridge	Tunnel
Pathologies:	Disorders list	Disorders list (see Chapter 4)
1. Initial conditions: Structures classification based on initial conditions	Initial Conditions, Structure characterisation	Degradation Potential Analysis (Influence Factors vs. disorders; frequency tables and GIS analyses)
2. Degradation (Rate): Main pathology scenarios identification based on service life conditions	Present Conditions Matrix + Degradation Matrix + Influence Factors Matrix	Degradation Rate Analysis (Correspondence Analysis: disorder type vs. influence factor modality/ attribute, GIS Analyses, Probability evaluation)
3. Evolution: Main pathologies evolution simulation	Evolution Models (Markov Chains based on successive inspections results)	Degradation Models for selected scenarios (see Chapter 2 and Chapter 6)

Another possibility for assessing disorders evolution with time is based on monitoring measurements interpretation. As suggested by the OFROU guideline (CH5) [100] and already done by CFF for railway tunnels (CH7) [32] when a tunnel shows a particular disorder it is important to follow its evolution. Moreover, when a tunnel is excavated in evolutive rock masses, the follow up of stresses and strains evolution with time may reveal quite useful.

For example, monitoring the evolution of tunnel shape deformations may help in foreseeing lining failure. However, for Swiss road tunnels at present, though strongly suggested in case of evident disorders, profile monitoring is not included within recommended tunnel survey techniques during principal inspections. Today, lining deformation can be measured with a good precision simply by using laser scanner profilometer (e.g. 0.3 mm at a distance of 2 m, is the precision of the scanner 3D used by CETu) as it is already done for the majority of railway tunnels (CH7) [32] and by CETu also for French road tunnels (FR3) [29].

In other cases the equipment can include extensometers bored in the excavation contour together with pressure cells. An example is given in Chapter 7 (see Paragraph 7.1) which shows how, using measured displacements it is possible to estimate the stresses in the lining after about 10 years of tunnel operation. This example uses measurements performed in the Mont Russelin tunnel (Jura Canton, N16) since its construction, ended in 1991, and during about 10 years (i.e. last measurements in September 2000). Due to the particular geological conditions (i.e. the tunnel crosses the formations typical of the Jura Mountains), 8 monitoring sections of the tunnel were equipped with:

- four extensometers (Interfels GmbH),
- four pressure cells (Glötzel GmbH),

placed at tunnel crown, invert and side walls.

Tunnel shape deformation, due to the delayed convergences of the rock mass, has been described by using extensometers measurements. As the changes in tunnel shape are the consequence of asymmetric loading conditions, then, changes in the tunnel shape (i.e. shape ovalisation, from circular to egg shaped) have been analysed in terms of axial load (i.e. thrust N) and bending moment (M) originating in the tunnel lining respectively at tunnel crown and side walls. In order to validate the procedure, the calculated values have been compared with the pressure cells measurements. A good agreement is found, though, the results depends on the measurements precision. Again, this example shows the possibility of interpreting displacements monitoring results through a simple model for estimating stresses in the tunnel lining.

TUNNEL CONDITIONS EVALUATION ("GLOBAL VS. DISORDER INDEX"). For a complete maintenance planning, a life-cycle cost analysis can be made by comparing repair options in both short and long term rehabilitation. Indeed, according to [151], the maintenance politic may change the total amount of money necessary for guaranteeing an acceptable service and safety quality level of the tunnel. The analysis should integrate information about tunnel conditions together with a socio-economical evaluation mainly based on the role/position of the tunnel on the whole road network. Though this topic is not treated in this framework, some considerations can be done in the perspective of indexing actual tunnel conditions. As shown in Chapter 3 (Table 3.6) in several countries, the responsibles of road and railway network management already use specific indexes for describing tunnel concrete lining conditions (as suggested by [100]). Sometimes, as it is done by CETu, France [29], also the water influence is considered.

Following the procedure used in this chapter for identifying the main factors that influence tunnel degradation, and considering the results summarised in Section 5.2.3, a global mark, in spite of its simplicity, can not be considered as an exhaustive index for describing tunnel conditions. An interesting/alternative approach could be the use of several indexes describing tunnel conditions with respect to the identified disorders. Pathology location and severity (i.e. affected zone extension) could be used as an internal weight for each index. Moreover, the identification of main scenarios that determine degradation rate could help in focusing attention (and means) on major degradation processes. Finally, after a good diagnostic of the tunnel conditions, social and economical factors will determine the tunnel owner decision. This procedure can be resumed in three main steps:

I. tunnel initial condition evaluation to assess the degradation potential of each structure, considering the identified pathologies,
II. estimation of degradation rate based on identified pathology scenarios,
III. socio-economical evaluation of the structure [31]; [22] and cost benefit analysis.

5.5 Data sources

1. MétéoSuisse Office fédéral de météorologie et de climatologie - Federal Office of Meteorology and Climatology: http://www.meteosuisse.admin.ch
2. OFEV Office fédéral de l'environnement - Federal Office for the Environment: http://www.hydrodaten.admin.ch
3. OFROU Office fédéral des routes - Federal Roads Office: http://www.astra.admin.ch
4. OFS Office fédéral de la statistique - Federal Statistical Office: http://www.bfs.admin.ch
5. Swisstopo Office fédéral de la topographie - Federal Office of Topography: http://www.swisstopo.ch

6. Convergence-Confinement Method for Long Term Analysis

Tunnels evolve with time. The rock mass and the lining often exhibit delayed behaviour. Thus, on the long term, together with all initial conditions it is necessary to take into account also operational conditions and all modifications and degradations occurring to both the lining structure and the rock mass. The intensity of these modifications depends on the age of the structure, geological and hydrogeological conditions, tunnel depth and initial stress field in the rock mass, lining materials quality, operation and environmental conditions. As reported in [5], the most simple way to take into account the different pathologies that characterise tunnel evolution with time, is to assess the characteristic time of the various processes. The main goal of this chapter is to use degradation models presented in Chapter 2, for estimating long term conditions of tunnels affected by disorders as described in Chapter 5. Long term stability conditions are evaluated in the framework of the convergence-confinement method. It is clear that the use of closed form solutions needs the tunnel to respect several conditions and may result in some simplification. An interesting aspect of presenting the long term behaviour in the convergence confinement-method framework is that it is a simple way to understand the influence of the factors identified in the previous chapter on the long term stability of tunnels. The models used in this chapter should help people in understanding tunnel degradation processes effects and can be adapted also for more refined numerical analyses.

6.1 Methodology

By summarising the results of the previous chapters, it is possible to determine a specific methodology for taking into account the influence of time dependent evolution on tunnel equilibrium (Figure 6.1). When a tunnel is excavated and the existing rock mass is replaced by a lining structure the modification of the original conditions is evaluated by taking into account several factors:

- mechanical properties and rheological behaviour of both the rock mass and the lining structure,
- initial stress conditions,
- construction method.

The same features, as demonstrated in Chapter 5, together with information about operation conditions and detailed inspections results can be used for evaluating tunnel long term behaviour. Thus, after tunnel construction, tunnel conditions evaluation is the result of a constant updating procedure, similar to the observational method, which should take into consideration:

- all the aggressive agents including water action, which mainly contribute to tunnel weathering. These actions can be modelled as reported already in Chapter 2 by strength and or, in the case of tunnel lining, by thickness reduction,
- all the ageing factors which can be modelled by stiffness reduction or strain increase,
- all other actions that may change the tunnel initial conditions (e.g. pore water pressure redistribution, consolidation).

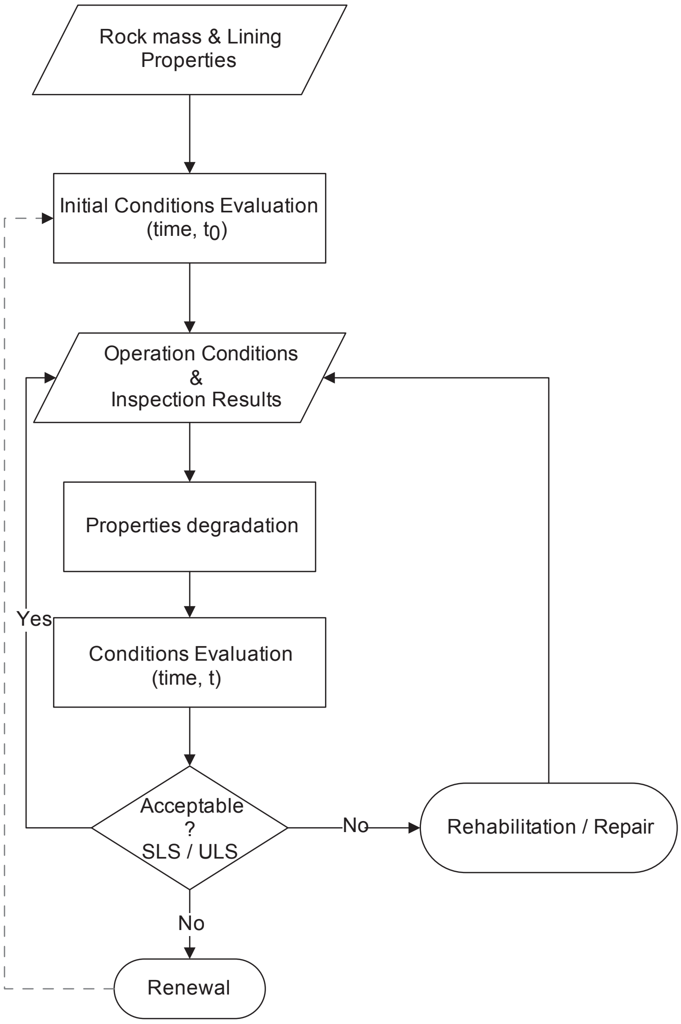


Fig. 6.1. Methodology for evaluating long term equilibrium condition (SLS = Serviceability Limit State; ULS = Ultimate Limit State).

After the construction, the equilibrium condition of the tunnel is modified due to its interaction with the environment together with the operational conditions. Moreover, due to the rheological behaviour of the rock mass and the lining, changes in stresses and strains may be observed in the tunnel and at the excavation contour. As shown in Figure 6.1, after initial conditions evaluation, it is necessary to determine/characterise how the rock mass and the lining properties change and recalculate the equilibrium of the structure under the new conditions.

All information related with tunnel operation conditions and disorders observed during principal inspections should be used for updating initial data and re-evaluating the tunnel equilibrium at regular intervals during its service life. The new conditions can be analysed in terms of Safety Factor (or Factor of Safety, FoS), by comparing the resistance (strength) of the lining structure and the effect of action (stress) in it. In the case of an axial-symmetric problem (assumption of the convergence-confinement method, Section 6.2.1), the Factor of Safety is more usually expressed as the ratio of the maximum pressure that the supporting structure can bear p_{max}, to the actual pressure exerted by the rock mass, p_{eq}:

$$FoS = \frac{p_{max}}{p_{eq}} \geq 1 \tag{6.1}$$

As it is described more in details in the following paragraphs, the resistance of the tunnel is a function of the supporting structure type and conditions, while the loads are mainly determined by the rock mass behaviour and conditions.

As previously shown (see Chapter 3) tunnel conditions evaluation can be done in a (semi-)qualitative way by inspection operators (i.e. quality indexes), or, in a more “accurate” way, by using numerical and analytical models (i.e. quantitative approach). It is interesting to observe that the general evaluation methodology, presented in Figure 6.1, may be applied to both qualitative and quantitative approaches including numerical modelling. Some applications will be presented in the following, in the framework of the convergence-confinement method.

6.2 The Cv-Cf method: Origins and development

As summarised by [40] in his PhD thesis, based on Kirsch’s solutions for cylindrical cavities, in 1938, Fenner developed a first closed form solution for stresses and strains distribution for a circular cavity excavated in an isotropic, homogeneous material with an elasto-plastic behaviour. In 1964, Pacher developed more solutions for different kinds of rocks and introduced the characteristic line for the rock mass. Through this graphical representation of the lining pressure versus the radial walls displacement, it was possible, for the first time, to describe the interaction between the excavated rock mass and the lining. Moreover, following the NATM concept (New Austrian Tunnelling Method), the contribution of the rock mass to the final stability conditions of the tunnel was finally taken into account.

Though, as identified by several authors during the seventies, the influence of the face and the tunnel excavation rate is a three dimensional problem, these difficulties were by-passed by developing simplified methods for taking into account the face effects in two-dimensional analyses. [115], for example, introduced a ‘fictitious’ internal pressure for describing the face effects in the vicinity of the analysed cross section.

The name “convergence-confinement method” (Cv-Cf) was given in 1978, during an AFTES meeting in Paris. The name of “characteristic lines” is due to the fact that the equilibrium of a tunnel, in plane strain conditions and for an axisymmetric problem, is described by the intersection of two curves:

1. the Convergence line (or Ground Characteristic Curve, GCC) which represents the rock mass response during excavation,
2. the Confinement line (or Support Reaction Line, SRL) which describes the supporting structure reaction.

A detailed literature review on the convergence-confinement method (Cv-Cf) has been given in [21]; [114]; [27]; [116].

6.2.1 Applicability & Notation

Though the analysis of rock mass/support interaction is a very complex problem and involves a lot of factors, in the framework of the continuum mechanics, the simplified approach proposed by the Cv-Cf method can be adopted for interpreting a certain amount of problems, in the applicability range defined by the following assumptions:

- circular tunnel,
- uniform (hydrostatic) stress field, which corresponds, before tunnel excavation, $\sigma_0 = \gamma_{rm} \cdot H$ to the overburden pressure at the tunnel depth, H,
- isotropic and homogeneous material (e.g. elastic or elasto-plastic),
- axial-symmetric supporting structure (e.g. closed concrete/shotcrete ring).

Notation

The geomechanical sign convention is applied, with compressive stresses assumed as positive. Also radial convergence is considered positive.

- H tunnel depth
- R tunnel radius

- σ_0 initial stress field
- σ_r radial stress
- σ_θ tangential stress
- $q = \sigma_\theta - \sigma_r$ deviatoric stress in cylindrical coordinates (axisymmetrical loading and plane strain conditions)
- t time
- p_i pressure acting on the tunnel lining
- pp_0 initial pore water pressure
- δ_r radial displacement, δ_R at the tunnel contour
- δ_{in} initial displacement at lining installation
- γ_{rm} rock mass unit weigth
- E Young's Modulus of the rock mass
- ν Poisson's ratio of the rock mass
- φ friction angle (Mohr-Coulomb criterion)
- c cohesion (Mohr Coulomb criterion)
- ψ dilatancy angle ($0 \leq \psi \leq \varphi$)
- E_c Young's Modulus of the concrete lining (E_s for the shotcrete support)
- ν_c Poisson's ratio of the concrete lining (ν_s for the shotcrete support)
- s_c lining thickness (s_s for the shotcrete support)
- f_c uni-axial compressive strength of the concrete (f_{c_s} for the shotcrete)
- k_s stiffness of the supporting structure
- p_{max} maximum pressure that the supporting structure can bear

6.2.2 Basic equations

Due to its simplicity, although limited by basic assumptions (see Paragraph 6.2.1), the Cv-Cf method is quite widespread and well developed and, as reported in [21]; [114], several authors proposed solutions for both the convergence and the confinement lines. Since the origins of the method until today, several types of rock mass and support have been considered. A summary of the method and basic equations for a circular tunnel with a concrete lining excavated in an elastic-plastic rock mass can be found in [126].

Convergence line

As previously introduced, the Ground Characteristic Curve (GCC) is the graphical representation of the relationship between radial convergence and internal pressure for a circular tunnel excavated in a medium subject to hydrostatic far-field stresses. The stress field changes during tunnel excavation and the maximum stresses are recorded at tunnel side walls. This zone may yield as described by means of a failure criterion. Thus, for drawing a GCC in the short term, it is necessary to define:

1. initial stress field (σ_0),
2. elastic mechanical parameters (E, ν),
3. failure criteria parameters (e.g. c, φ),
4. plastic dilatancy (ψ).

For the axisymmetric problem the equilibrium conditions of Equation 6.2 should always be satisfied:

$$\frac{\partial\sigma_r}{\partial r} + \frac{\sigma_r - \sigma_\theta}{r} = 0 \tag{6.2}$$

Moreover, the deformation can be expressed as a function of the radial displacements, δ_r:

$$\varepsilon_r = \frac{\partial\delta_r}{\partial r} \tag{6.3}$$

$$\varepsilon_\theta = \frac{\delta_r}{r} \tag{6.4}$$

As reported in [21]; [114], several authors proposed closed form solutions for the GCC. If the rock mass behaves as an elastic-plastic material, as long as there's no failure around the tunnel (i.e. the internal pressure, p_i, remains bigger than the critical pressure, p_{cr}) the rock mass behaviour is described by Hooke's law. Thus, it is possible to derive both strains and stresses according to the Lamé's solutions for a cylindrical symmetry. When the failure is reached, the internal pressure becomes smaller than the critical pressure and a plastic zone appears at the tunnel contour. The radius of this zone, R_{pl}, compared to the tunnel radius, R, together with stress and strain distribution depend not only on the initial stress, σ_0 and the internal pressure, p_i but also on the parameters chosen for the rock mass according to the failure criterion (see Figure 6.2).

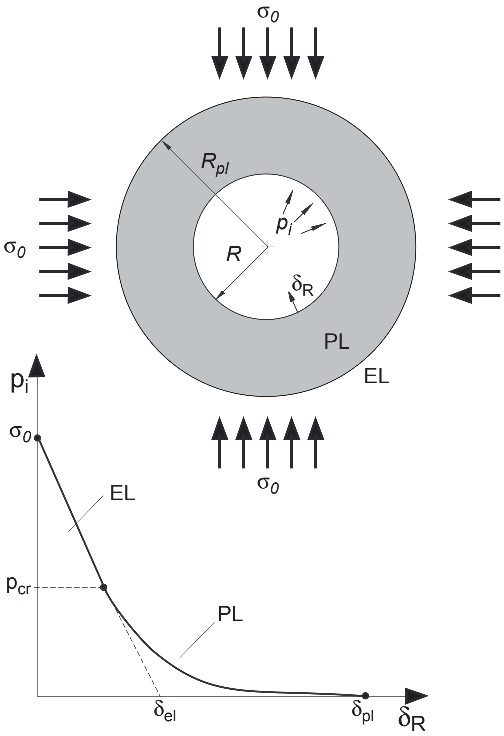


Fig. 6.2. GCC for a tunnel excavated in a rock mass with elastic-plastic behaviour.

Confinement line

The confinement line represents the relationship between supporting structure radial displacements and uniform pressure applied to the tunnel extrados. In the Cv-Cf approach, the load that the lining should bear is mainly a function of the rock mass displacements. For drawing the confinement line (Figure 6.3) two main characteristics are needed:

1. stiffness, k_s
2. maximum pressure (yielding pressure), p_{max}.

Several equations have been proposed in the past by considering different types of support [114]. When several supports are installed the total stiffness of the system is the sum of the stiffnesses of each support.

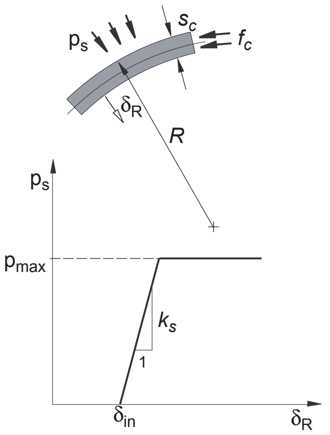


Fig. 6.3. Confinement line (SRL) for a concrete ring (elastic-perfectly plastic behaviour).

The elastic part of the support reaction line can be drawn by using the following equation:

$$p_s = k_s \cdot \left(\frac{\delta_R - \delta_{in}}{R}\right) \tag{6.5}$$

where k_s is the supporting structure stiffness, p_s the pressure at the extrados, δ_R the radial displacement at the tunnel wall and δ_{in} the initial convergence of the excavation wall before the support is placed. A correct evaluation of the equilibrium between the rock mass and the lining needs a good estimation of δ_{in} which depends on 3D face effects and on lining laying distance. For example, it can be evaluated by using the similarity principle introduced by [35]. By knowing the distance of the section where the support is constructed from the tunnel face D_0 and the similarity ratio χ between the final convergence of an unlined tunnel excavated in an elastic-plastic medium δ_{pl} and in an elastic medium δ_{el} (see Figure 6.2), δ_{in} can be evaluated as follows:

$$\delta_{in} = \chi \cdot f\left(\frac{D_0}{\chi}\right) \tag{6.6}$$

where

$$f(D) = \frac{1+\nu}{E} \cdot \sigma_0 \cdot R \cdot \left[0.29 + 0.71 \cdot \left(1 - e^{-1.5 \cdot \left(\frac{D}{R}\right)^{0.7}}\right)\right] \tag{6.7}$$

and

$$\chi = \frac{\delta_{pl}}{\delta_{el}} \tag{6.8}$$

Also the gap between the rock mass and the temporary support should be considered as an additional displacement.

6.2.3 Time dependent behaviour and degradation

Since the origins of the Cv-Cf method, time was identified as an important factor to be considered for evaluating properly the tunnel equilibrium conditions. Thus, as summarised in Table 6.1, several authors tried to include time and degradation effects in the calculation of the characteristic lines. As it is possible to see most authors introduced time by changing the rheological behaviour of the rock mass. This is mainly due to the fact that this method can be used for interpreting convergence measurements during tunnel construction, when, under normal conditions, the major delayed phenomena depend on the rock mass and not on the lining evolution with time.

Tab. 6.1. Summary of existing solutions, in the convergence-confinement method, considering the time effects on the rock mass behaviour. For each degradation process the list is given in a chronological order.

[Reference]	Constitutive Model (Rock mass)	Special Feature (long term)
	ROCK MASS - CONVERGENCE LINE	
Ageing:		
[112]	viscoelastic (Kelvin Voigt model)	Young's Modulus reduction with time
[81]	elastic- non linear viscoelastic	creep law distinguishing time and stress effects: primary creep (both power and logarithmic laws) and secondary creep (steady-state)
[15]	elastic-viscoplastic (Bingham Norton model)	two solutions for viscoplastic behaviour of the excavated rock mass based on the lining stiffness
[57]	elastic-viscoplastic (Bingham Norton model) with peak and residual conditions in plasticity (Mohr-Coulomb)	semi-analytical solution by considering a plastic criterion (peak and residual conditions)
[36] [37] [38] [39]	linear viscoelastic, linear elastic-linear viscoelastic, linear elastic-non linear viscoelastic, elastic-viscoplastic	constitutive model for a horizontal tunnel, with and without supporting structure
[140] [139]	elastic perfectly plastic with softening (Mohr-Coulomb) / linear viscoelastic behaviour (Kelvin Voigt model)	modelling creep distinguishing time and face effects time with logarithmic law
[13]	elastic plastic with softening	modelling rock mass squeezing behaviour by using strain softening law
[125]	elastic-plastic & viscoplastic with softening (Mohr-Coulomb)	time dependent softening after passing the limit of viscoplastic strain
[95]	elastic viscoplastic (i.e.'non linear viscoelastic')	creep law by separating time and stress effects: secondary creep (steady-state) described by Norton's law & primary creep described by Lemaitre's law (time power law)
[23]	elastic perfectly plastic (Mohr-Coulomb) without dilatancy	swelling law by Gysel, (1987)
[17]	elastic-viscoplastic (i.e.'non linear viscoelastic')	primary creep modelled with Lemaitre's law distinguishing time and stress effects
Weathering:		
[80]	elastic-plastic	strength properties reduction with time (isochrone curves)
[40]	elastic plastic with softening (Mohr-Coulomb)	strength reduction effects on support system loading
[26]	elastic plastic with softening (Mohr-Coulomb & Hoek-Brown)	rock mass strength and stiffness properties reduction
Other Actions:		
[58]	elastic plastic (Mohr-Coulomb) (dilatancy $\psi = 0$ or $\psi = 0.5 \cdot \varphi$)	redistribution of pore water pressure after tunnel construction (New Design Method)

[Reference]	Constitutive Model (Rock mass)	Special Feature (long term)
	LINING - CONFINEMENT LINE	
[107] [108]	elastic plastic (Mohr-Coulomb)	modelling progressive hardening of shotcrete during tunnel construction

Convergence line

AGEING. Rock mass ageing has been introduced by several authors using different rheological models: Maxwell and Kelvin Voigt [112], Bingham [15], Rousset [125]. In 1980, under the hypothesis of a time function independent of stresses, [81] modelled the creep behaviour of the rock mass by a non linear viscoelastic model. A similar approach has been used by [140] for simplifying the convergence measurements interpretation during tunnel excavation. As the tunnel side walls still continued to converge when the face was stopped, they separated rock mass rheological behaviour (i.e. time effects) from the face advancing effects. Thus, according to this approach, when the tunnel excavation is completed, the rheological behaviour of the rock mass may still contribute to changing the final equilibrium conditions of the tunnel. By introducing a new general constitutive law, [38]; [39] developed several parametric analyses for a circular excavation. Both viscoelastic and viscoplastic behaviours of the rock mass were considered.

Without considering the time-dependent behaviour of the tunnel due to face advancing, [15] analysed the viscoplastic behaviour of the excavated rock mass by considering two different types of lining. As expected, the rock mass final convergence is a function of the lining stiffness: for a smaller stiffness the rock mass strains increase. Under the condition of non compressible material, [95] proposed an approximated solution for describing the steady state creep by means of the Norton's law. In his PhD thesis, [17] used the same solution for evaluating viscous strains in the excavated rock mass induced by primary creep as described by Lemaitre's law. For what concerns viscoplastic models further consideration should be done. Due to its formulation, the viscoplastic model as proposed by [95]; [17] does not take into consideration the rock mass failure criterion. In fact, it rather corresponds to a non linear viscoelastic model according to the definition of [81]. As a matter of fact, a complete formulation for describing elastic viscoplastic behaviour requires strength parameters as shown by [125]. For modelling the viscoplastic behaviour of the excavated rock mass he introduced a new model described by ten parameters: five for defining the strain softening behaviour by means of a viscoplastic criterion (i.e. three limit strains $\varepsilon_0, \varepsilon_1, \varepsilon_2$ and two cohesions, C, C_0 for describing peak and residual conditions), two viscous parameters (η, viscosity and n, stress power), two elastic parameters (E and ν) and the friction angle (φ). [57] developed a complete semi-analytical solution for a circular tunnel excavated in an elasto-viscoplastic rock-mass with a Mohr-Coulomb criterion. The viscoplastic strains develop only in the plastic zone around the excavation.

Within the tunnel ageing pathologies described in Chapter 2, it has been considered also the squeezing behaviour of the rock mass. [13] suggested to introduce strain softening laws in the Cv-Cf analyses for modelling the long term behaviour of tunnel in squeezing rock masses. Thus, according to this approach, on the long term the plastic zone around the tunnel is characterised by residual strength parameters.

Finally, swelling behaviour has been modelled by [23]. As detailed in Chapter 2 (from Equation 2.24 to Equation 2.28), according to the simplified approach proposed by Gysel, (1987), the swelling behaviour of a rock mass induces volumetric strain proportional to the ratio between the mean stress and the swelling limit p_g. Anyway, as observed by [8] a correct interpretation of the swelling behaviour requires numerical solutions for taking into account hydro-mechanical coupling.

WEATHERING. Based on uniaxial compression tests performed by several authors, [80] stated that the rock mass mechanical characteristics may reduce on the long term due to weathering. By solving the GCC equations using long term parameters he drew several isochrones corresponding to a given level of properties decrease, and, thus, to a certain time after tunnel excavation. Unfortunately, though clearly identified, the time dependency of the mechanical parameters decrease, is not properly described by any equation and may change from tunnel to tunnel. [40] in the framework of the Cv-Cf method developed a numerical solution for evaluating the change in support loading conditions due to rock mass strength properties reduction with time caused by weathering agents. Based on the theory of the strength and stiffness properties reduction, assuming rock mass weathering proportional to the plastic volumetric strain around the tunnel, ε_v^p , [26] proposed two solutions for a tunnel excavated in elastic plastic rock mass with softening behaviour considering both Mohr-Coulomb and Hoek-Brown criteria.

OTHER ACTIONS. As observed in Chapter 2, when tunnels are driven in water saturated rock masses, it is necessary to take into account the pore water pressure redistribution with time. By considering this redistribution of pore water pressure in low permeable saturated porous media, [58] in his PhD thesis, proposed a new design method which allows to evaluate the long term pressure acting on tunnel lining as a function of the water boundary condition at the gallery wall (permeable or impermeable lining).

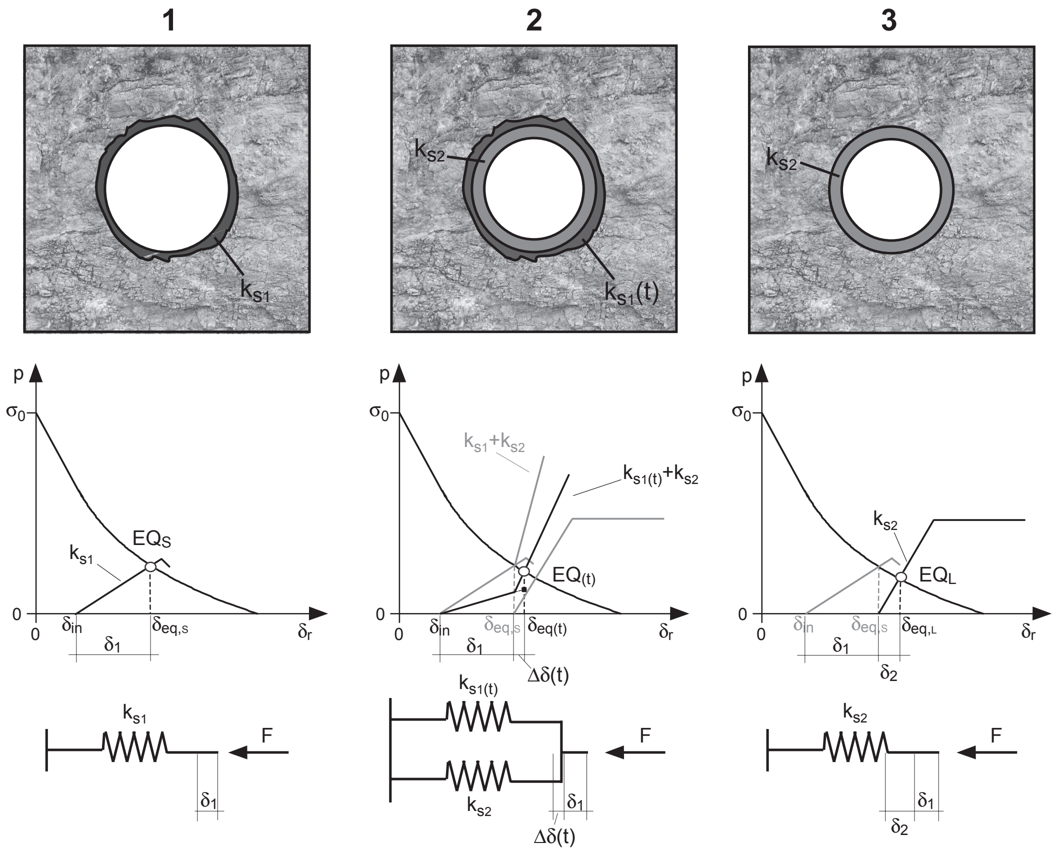
Confinement line

As the equilibrium pressure between the rock mass and the lining is found by the intersection of both characteristic lines, any time evolution of the lining characteristics will result in changes in the confinement line and in the equilibrium point. Now, as reported in Table 6.1, for what concerns the confinement line, very few attempts have been done for introducing time effects and taking into account long term changing conditions. [108]; [107] proposed a method for evaluating transient conditions during hardening of shotcrete support and representing this time evolution using confinement line. By means of an iterative process the progressive increase of both elastic modulus and strength of shotcrete with time is evaluated during the construction.

Some considerations should be done for explaining the changing conditions of the support/lining interaction with the excavated rock mass. Primary support is applied immediately after tunnel excavation to ensure safe working conditions and to mobilize rock mass strength by controlling displacements. Then, the construction is completed and a definitive lining is constructed. As when several supports are installed the total stiffness of the system is represented by the sum of the stiffnesses of each support, the same consideration can be done when the lining is installed. The new equilibrium depends on the total stiffness, on the displacement occurred before placing the definitive lining and on the displacement of the supporting structure at the moment of the lining installation. Due to the corrosive action of ground water and soil, the action of temporary support is neglected on the long term. As a matter of fact, in the majority of tunnels the waterproofing membrane is placed between the temporary support and the definitive lining, thus, the support degrades much faster than the lining does. Figure 6.4 show the three steps that determine the confinement line with time:

1. At the beginning the equilibrium depends only on the support characteristics.
2. Then, when the lining is completed, both temporary support and lining work together with a progressive degradation of the temporary support.
3. On the long term, only the lining contributes to tunnel stability.

As observed by [114], the support behaviour changes from type to type. For example, a temporary support composed by steel sets will show an elastic perfectly plastic behaviour characterised by a considerable ductile strain if compared to a shotcrete ring (without fibres). The maximum load that the supporting structure can bear is evaluated by considering the material strength. When one element of the system is yielded its contribution can be considered until its final failure. Thus, the maximum load depends on the sum of the final strength of all elements that contribute to tunnel equilibrium and on their deterioration with time. As written previously, construction information is necessary for better evaluating the tunnel equilibrium conditions in the different periods of the tunnel life.



$F = k_{S1} \cdot \delta_1$
$\delta_1 = \delta_{eq,S} - \delta_{in}$

$F = k_{S1}(t) \cdot (\delta_1 + \Delta\delta(t)) + k_{S2} \cdot \Delta\delta(t)$

$F = k_{S1}(t) \cdot \delta_1 + (k_{S1}(t) + k_{S2}) \cdot \Delta\delta(t)$

$F = k_{S2} \cdot \delta_2$
$\delta_2 = \delta_{eq,L} - \delta_{eq,S}$

Fig. 6.4. Support / Lining system: changing equilibrium conditions with time (a numerical example is described in Section 6.3).

6.3 Long term Cv-Cf analyses

In Chapter 5, it was shown how, after several years of operation, main tunnel pathologies depend on several factors that determine both initial conditions and tunnel degradation rate. As, tunnel conditions evolve with time, the safety and the serviceability of the tunnel must be checked during all its life (Figure 6.5):

- at short term, during/after construction,
- at long term, during operation.

For this purpose, it is necessary to estimate how the equilibrium conditions may change when time effects affect the excavated rock mass and the supporting structure.

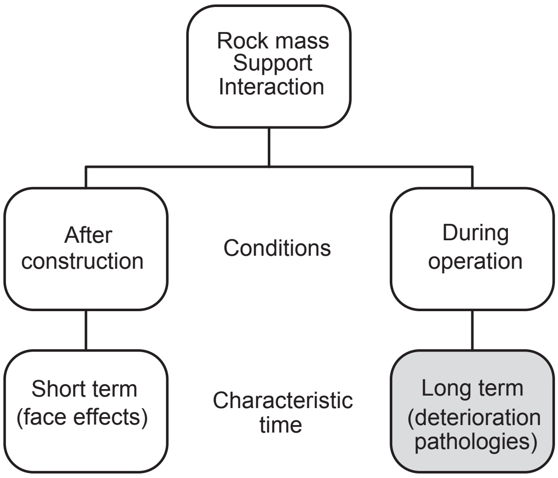


Fig. 6.5. Interaction between rock mass and support/lining structure: short term vs. long term.

Based on the recurrent disorders identified in the previous chapter, the following paragraphs show some examples for assessing the influence of the delayed behaviour on the global stability of tunnels. Typical tunnel pathologies can be modelled by changing mechanical properties, main features, and/or behaviour of both the rock mass and the lining. Though limited by its basic assumptions, the convergence-confinement method can be considered as a simple tool for approaching the complex problem of tunnel long term degradation. Indeed, as already explained in Section 6.2 (e.g. Table 6.1), by introducing the above mentioned changes in the Cv-Cf analyses, tunnel equilibrium (i.e. the intersection point between the GCC and the confinement line) evolves with time as shown in Figure 6.6.

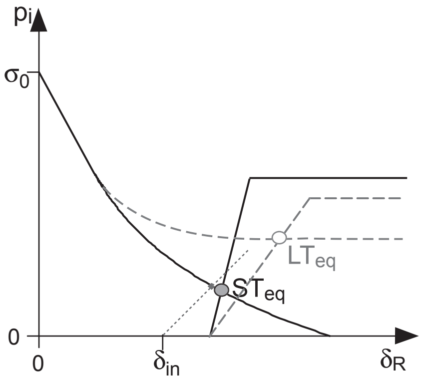


Fig. 6.6. Short and long term equilibrium conditions, in Cv-Cf graphical representation (note that the short term equilibrium ST_{eq}*, corresponds to the definitive lining equilibrium* EQ_L *of Figure 6.4).*

Several degradation models (already described in Chapter 2) have been integrated in Cv-Cf analyses for estimating the effects of time dependent pathologies on tunnel stability conditions. According to the time dependent representation introduced by [80], isochrones are represented showing the changing conditions from short to long term. In some cases it has been possible to represent the curves 10 years after construction and each 20 years until the end of tunnel service life (i.e. about 90 years). While, when there's no explicit time law the curves are represented only in short and long term conditions. Then, the results are analysed in terms of Safety Factor, as introduced in Section 6.1.

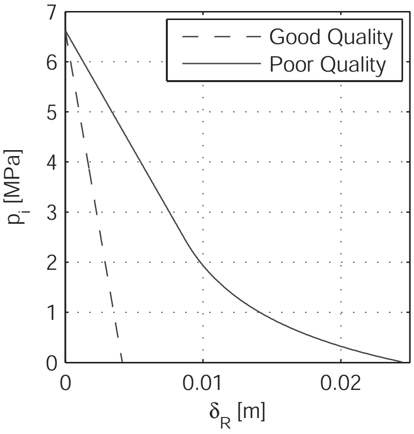
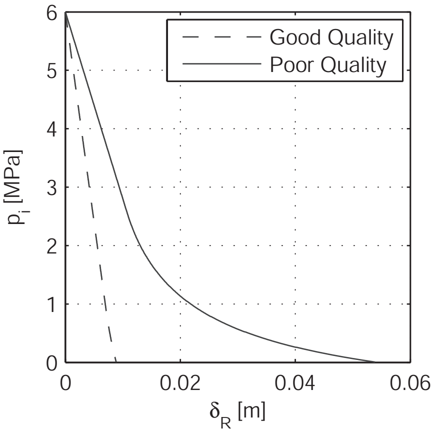
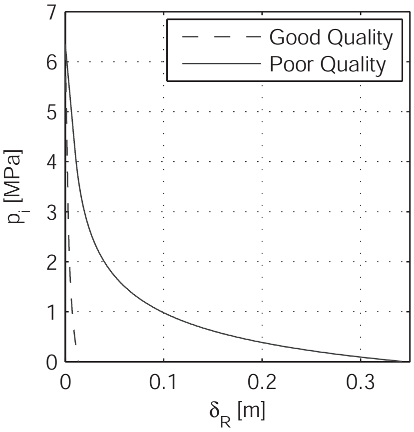
In the following analyses, for an easier comparison of the results, only one set of parameters has been considered for the tunnel structure:

- Tunnel radius, $R = 5$ m (circular section)

- Tunnel depth, $H = 250$ m (i.e. $H \gg R$, for respecting the assumption of hydrostatic stress field)
- Temporary support installed at $D_0 = 2$ m from the cutting face: 0.25 m thick Shotcrete with $E_s = 23$ GPa, $\nu_s = 0.2$, $f_{c_s} = 14$ MPa, (as the temporary support should guarantee the tunnel stability during construction, at a very short term, the mechanical characteristics are lower than the respective values for shotcrete after 28 days)
- Definitive lining: 0.3 thick concrete ring with $E_c = 35$ GPa, $\nu_c = 0.2$, $f_c = 40$ MPa. According to the Model Code 1990 [28], the lining behaves as an elastic perfectly plastic material (ductile strain $\varepsilon_u = 0.4\%$ for a normal concrete with a compressive strength of about 30-40 MPa).
- Waterproofing system placed at the definitive lining extrados (i.e. between the temporary support and the definitive lining).

As tunnel degradation potential and rate, and disorder type, depend on geological conditions, three typical Swiss rock formations have been considered to model different types of delayed behaviour. For each formation a good and a poor quality set of mechanical parameters is used, for respectively massive and fissured rock masses (Table 6.2).

Tab. 6.2. Mechanical parameters for three different Swiss rock formations with respectively good and poor qualities.

Good Quality	**Poor Quality**	**GCC Short Term Conditions**
CARBONATES		
$\gamma_{rm} = 26.5\ \mathrm{kN/m^3}$	$\gamma_{rm} = 26.5\ \mathrm{kN/m^3}$	
$E = 10'000$ MPa	$E = 3'000$ MPa	
$\nu = 0.25$	$\nu = 0.25$	
$\varphi = 40°$	$\varphi = 30°$	
$c = 4.5$ MPa	$c = 2$ MPa	
SANDSTONES		
$\gamma_{rm} = 24\ \mathrm{kN/m^3}$	$\gamma_{rm} = 24\ \mathrm{kN/m^3}$	
$E = 4'500$ MPa	$E = 2'000$ MPa	
$\nu = 0.25$	$\nu = 0.25$	
$\varphi = 35°$	$\varphi = 30°$	
$c = 2$ MPa	$c = 0.5$ MPa	
MARLS		
$\gamma_{rm} = 25.4\ \mathrm{kN/m^3}$	$\gamma_{rm} = 25.4\ \mathrm{kN/m^3}$	
$E = 8'000$ MPa [a]	$E = 1'500$ MPa	
$\nu = 0.4$	$\nu = 0.4$	
$\varphi = 20°$	$\varphi = 15°$	
$c = 1$ MPa	$c = 0.5$ MPa	

[a] This value, though it may seems quite high for common marls (usually characterised by smaller values, i.e. $E \sim 3'000$ MPa), is valid for silt-marls of the Jura and comes from interpretation of in situ measurements, as reported in [18].

As it is possible to see, according to the type of rock formation, in the case of unsupported tunnels, the final convergence can be very different. Also the rock mass quality seems to play an important role in determining the final side walls displacement.

6.3.1 Long term convergence line

As observed by several authors (Table 6.1), the rock mass delayed behaviour may result in significant tunnel convergences even a long time after the tunnel construction and cause severe damages to the tunnel lining if it is not taken into account during tunnel lining design. With reference to Table 2.3 of Chapter 2, it is possible to distinguish between:

- Ageing, physical changes which can be modelled by increasing strains,
- Weathering and chemical changes, which can be modelled by reducing strength,
- Other actions, as, for example, consolidation, which can be modelled by dissipation of pore water pressure in the ground mass with time.

Ageing

As described in Chapter 2, ageing of rock masses mainly depends on the rheological behaviour of the solid skeleton. As already mentioned in [43] the rock mass rheological behaviour may be approximated by using several rheological models composed by springs (Hooke elements - elastic), sliders (St.Venant elements - plastic or frictional cohesive) and dashpots (Newton elements - viscous). A comprehensive presentation of existing models has been given by [38]; [82]; [13]. As summarised in Table 6.1, the time dependent behaviour of the excavated rock mass has been described in the past, mainly by using two classes of models:

- elastic-viscoelastic (Figure 6.7 left),
- elastic-viscoplastic (Figure 6.7 right).

The main difference between the two classes is the strain developed on the long term. Viscoplastic materials show permanent strains whose rate is a function of the deviatoric stress. According to [120]; [82], sedimentary and metamorphic rocks (i.e. limestones, sandstones, schists, clays, marls) often exhibit a viscoplastic behaviour with significant strains that may last long term after the tunnel or cavern excavation, while hard igneous rock or intact calcareous rock masses usually show viscoelastic behaviour with small deformations. Thus, rheological behaviour depends on rock microstructure. As proposed by several authors mainly based on empirical/experimental results [43], viscoelastic models can be used for rocks with a “crystalline behaviour” as: granite, gneiss and intact carbonates. Viscoplastic models better describe the long term behaviour of tunnels excavated in marl, clays, schists and phyllites.

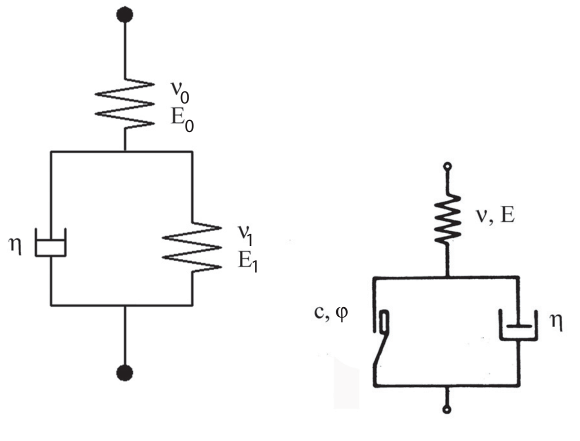


Fig. 6.7. On the left, elastic-viscoelastic model: a spring in series with a Kelvin-Voigt Model (a spring and a dashpot in parallel). On the right, elastic-viscoplastic model: a spring in series with a Bingham-Norton Model (a dashpot and a slider in parallel).

VISCOELASTICITY. Though [77] reports that some authors find the viscoelastic laws not appropriate for rock masses, several attempts of interpreting delayed convergences of excavated tunnels used in the past viscoelastic models. The most widespread model for representing the viscoelastic behaviour of the rock mass is the Kelvin-Voigt model. It is composed by a spring in series with a parallel of a spring and a dashpot (Figure 6.7 on the left). This model has been used, for example, for predicting convergences of chalk during Channel Tunnel construction [113].

For drawing the GCC in short and long term conditions, it is necessary to consider different values of Young's Modulus (i.e. spring) in short and long term conditions and introduce the viscosity of the rock mass (i.e. dashpot). The resulting radial displacements are described by the following exponential curve:

$$\delta_R(t) = \delta_{t_0}^{el} + \frac{\sigma_0 \cdot R}{2 \cdot G_1} \cdot \left(1 - e^{\left(-\frac{t}{T}\right)}\right) \tag{6.9}$$

with

- $\delta_{t_0}^{el} = \dfrac{\sigma_0 \cdot R}{2 \cdot G_0}$ elastic displacement at $t = 0$, where $G_0 = \dfrac{E_0}{2 \cdot (1 + \nu_0)}$ is the short term shear modulus
- $T = \dfrac{\eta}{G_1}$ the relaxation time, which is an estimation of the creep speed and is related to time dependent properties of the rock mass (η is the viscosity and G_1 is the long term shear modulus).

If the temporary support is placed at time t_0 after the tunnel excavation, when the tunnel-support system reach is elastic equilibrium, which is supposed to be instantaneous, the definitive lining is constructed. Then, with time passing, the load that the lining has to bear changes and can be estimated as follows [113]:

$$p_s = \frac{k_s}{k_s + 2 \cdot G_\infty} \cdot \left(1 - \frac{G_0}{G_\infty}\right) \cdot e^{\left(-\frac{t_0}{T}\right)} \cdot \left(1 - e^{\left(-\frac{t - t_0}{T}\right)}\right) \cdot \sigma_0 \tag{6.10}$$

where $G_\infty = \dfrac{G_0 \cdot G_1}{G_1 + G_0}$.

If time effects are neglected during tunnel excavation, it is possible to draw a short term GCC, then, after evaluating the long term pressure and displacement by introducing viscoelastic parameters it is possible to draw the long term GCC. Among the rock formations proposed in Table 6.2, according to the previous considerations, it has been chosen to represent the long term viscoelastic behaviour of a tunnel excavated in carbonates of good quality. According to values found in the literature, the viscous behaviour has been characterised by choosing $T = 30$ days. Figure 6.8 shows how the tunnel equilibrium conditions evolve with time.

As it is possible to see, the viscoelastic behaviour has a very short characteristic time (i.e. T) compared to the tunnel service life. This results in a significant change of the equilibrium point at the beginning of the tunnel service life, followed by a progressive stabilisation with time passing. Thus, under the hypothesis that the lining does not show any delayed behaviour, the long term conditions are already established at a very short period after the tunnel construction, as shown by the FoS evolution (see bottom of Figure 6.8). By considering a longer characteristic time (i.e. $T = 75$ years), as shown in Figure 6.9, it takes more time to the tunnel for reaching its final equilibrium.

The main problem that may be identified with this kind of model is the strong limitation in representing the real evolution of the rock mass behaviour with time. As a matter of fact, the hypothesis of viscoelasticity cannot take into consideration the development of a damaged zone around the excavation where, also in case of very good rock formations, plastic strains may arise.

The high initial value of the Factor of Safety, represented in Figure 6.8 can be quite surprising, but it is essential to keep in mind that is related to the ideal condition of a concrete ring subjected to an axisymmetric load. Though in reality this condition is rarely met, it is important to notice that the final conditions show a remarkable decrease of Safety Factor which should be taken into consideration in tunnel design and maintenance.

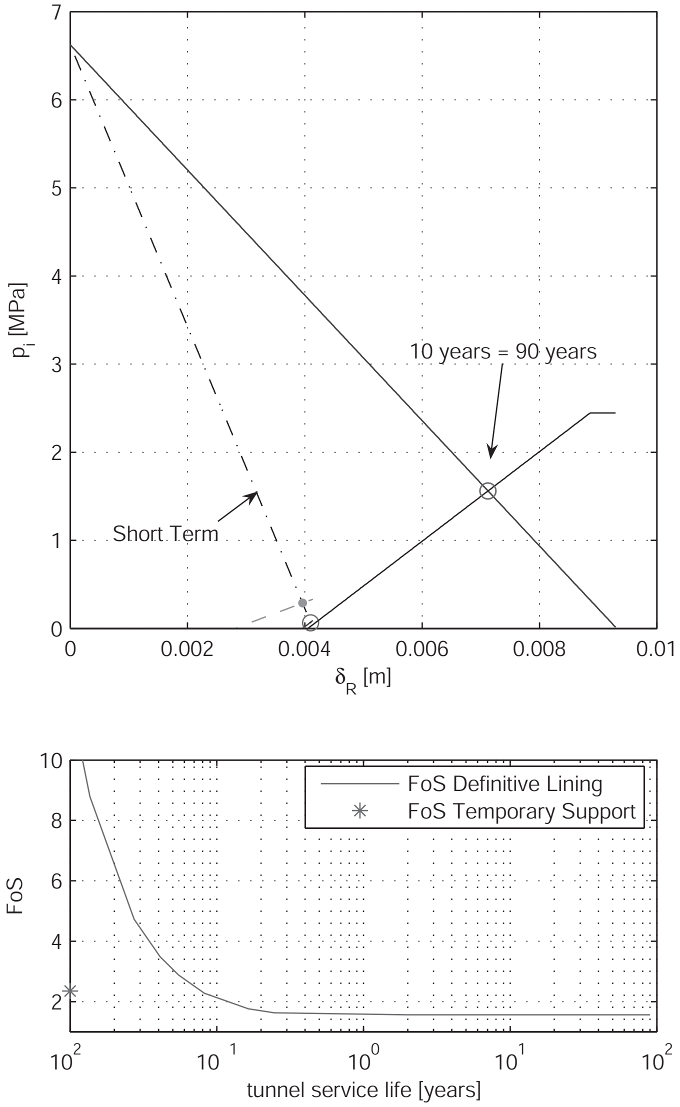


Fig. 6.8. On top: short and long term equilibrium conditions for a tunnel excavated in a viscoelastic rock mass (CARBONATES, good quality parameters, Table 6.2; $T = 30\text{days}$). The GCC 10 years after tunnel construction is almost equal to the 90 years one. On bottom: FoS evolution with time (the logarithmic scale has been chosen for showing that the main changes in the equilibrium conditions occur at the beginning of the tunnel service life).

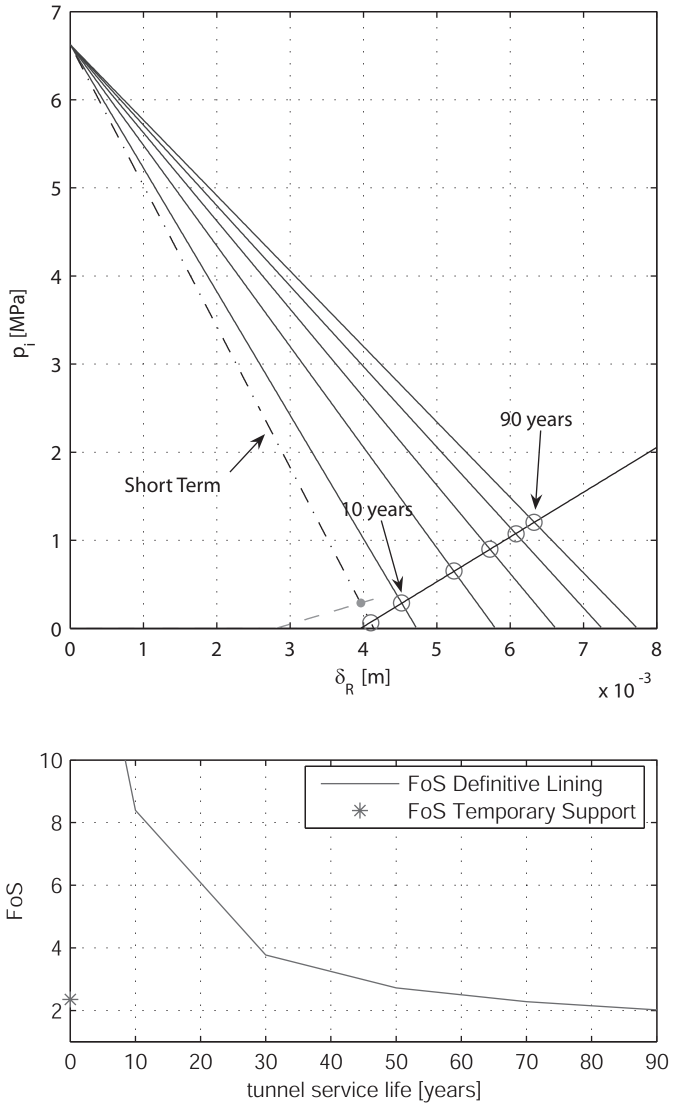


Fig. 6.9. On top: short and long term equilibrium conditions for a tunnel excavated in a viscoelastic rock mass (CARBONATES, good quality parameters, see Table 6.2; $T = 75\ \text{years}$). The GCC is represented after tunnel construction and during tunnel service life each 20 years from 10 years till to 90 years. On bottom: FoS evolution with time.

VISCOPLASTICITY. As reported in Table 6.1, several authors tried, in the past, to represent the delayed convergences of the rock mass by introducing a viscoplastic behaviour. As a matter of fact, the viscoplastic behaviour requires a plastic criterion and the number of parameters necessary to model this behaviour can become quite important (e.g. [125]). To simplify the problem, as shown in Table 6.1, some authors did not consider a plastic criterion in their model of the rock mass and adopted a non linear viscoelastic behaviour as defined by [81]. For sake of simplicity these models will however be referred to as viscoplastic models.

The elasto-viscoplastic behaviour is usually represented by a Bingham-Norton model, shown in Figure 6.7 right. The Hooke element, which represents the instantaneous elastic behaviour of the rock mass, is characterised by Poisson's ratio ν and Young's Modulus E. The plastic behaviour is associated to a slider while the viscosity is represented by a dashpot. The total strain rate is equal to the elastic strain rate plus the viscoplastic strain rate:

$$\dot{\varepsilon} = \dot{\varepsilon}^{e} + \dot{\varepsilon}^{vp} \tag{6.11}$$

As already introduced in Chapter 2, the time-dependent behaviour of a rock formation can be described by a creep test: if the deviatoric stress is high enough the primary creep evolves to a secondary creep (steady state) and finally to a tertiary creep (failure) with a progressive damage of the material.

As reported by [39] in most cases the rock mass does not attain a fully steady state condition (i.e. secondary creep, as described in Chapter 2). For modelling primary creep, Lemaitre evaluated the viscoplastic strains by considering only the deviatoric part of the stress tensor as responsible of viscoplastic strains. Starting from Lemaitre's law, [17] evaluates the viscoplastic strain ε_{vp}, for cylindrical cavity (see also Equation 2.20 and Equation 2.22, Chapter 2):

$$\varepsilon^{vp} = a_{cyl} \cdot (q)^{\beta} \cdot t^{\alpha} \tag{6.12}$$

where t is the time, $q = \sigma_{\theta} - \sigma_r$ the deviatoric stress under the assumption of axisymmetric loading, in plane strain conditions, and for an incompressible material, i.e. $\nu = 0.5$,

$$a_{cyl} = (0.75)^{\frac{\beta + 1}{\alpha}} \cdot \left(\frac{A}{\alpha}\right)^{\alpha} \tag{6.13}$$

and where α and β are two material constants mainly depending on the deformation mechanism (e.g. dislocation or glide), and A on the temperature (see Chapter 2). Moreover, under the same conditions, it is possible to assume that $\varepsilon_{vp_r} + \varepsilon_{vp_{\theta}} = 0$. Under these assumptions [17] evaluates the deviatoric stress in transient conditions using the approximate analytical solution proposed by [95]. Moreover, using this solution, the long term GCCs were drawn as detailed in [126].

An example of evolution of wall displacement and pressure with time has been calculated using viscoplastic parameters for the Mont Terri tunnel (Jura Canton, Switzerland). This tunnel crosses the Aalenian marls characterised in Table 6.2 (i.e. MARLS, good quality). Two sets of parameters have been compared (see Table 6.3). The first, proposed by [75], resulted by laboratory creep tests conducted at LMR-EPFL, while the second, proposed by [18], was evaluated by curve fitting of in situ convergence measurements.

Tab. 6.3. Lemaitre's parameters for the Mont Terri Aalenian marl as proposed by [75] fitted on creep tests and [18] fitted on in situ convergences.

Parameter	**[75]**	**[18]**
A	2E-43	3E-61
α	0.083	0.07
β	1.1	1.1

The comparison of Figure 6.10 and Figure 6.11 shows that the final displacements may change a lot from one set to the other one. Anyway, it can be considered that the set of parameters provided by curve fitting of convergence measurements better describes the tunnel behaviour on the long term.

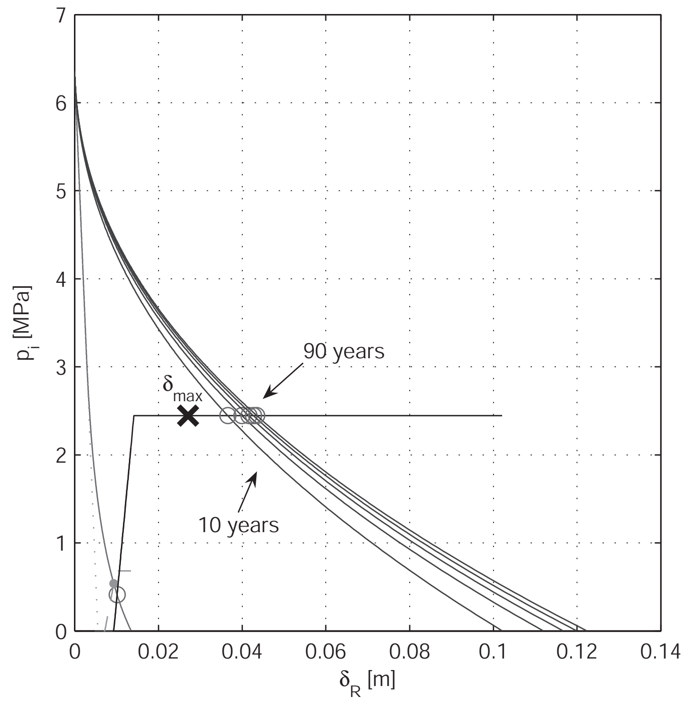


Fig. 6.10. Convergence-confinement analysis with an elastic-viscoplastic rock mass. Parameters after [75]. The GCC is represented after tunnel construction and during tunnel service life each 20 years from 10 years till 90 years. The ductility of the lining does not satisfy the long term equilibrium.

Figure 6.10 shows in a Cv-Cf diagram the evolution with time of the radial wall displacement for a tunnel excavated in a rock mass with a viscoplastic behaviour, using the parameters proposed by [75]. As it is possible to see, the lining strength is exceeded after less than 10 years of service life. Note that on delayed behaviour of the lining is considered during all tunnel service life. In reality the mechanical properties of concrete slightly increase with time and this phenomenon may reduce the possibility of lining to fail. But, at the same time, when the lining deterioration rate is faster than the improving rate the Safety Factor starts to reduce again.

In reality, according to constructive data about the Mont Terri tunnel available in the TDB, the part of the tunnel crossing Aalenian marls has a bigger radius (i.e. $R = 6$ m) and a thicker definitive lining. Thus, as shown in Chapter 7 (Section 7.2), another analysis has been done considering the real tunnel geometry together with a reinforced concrete definitive lining of 60 cm. In this case, the lining strength is far from being reached.

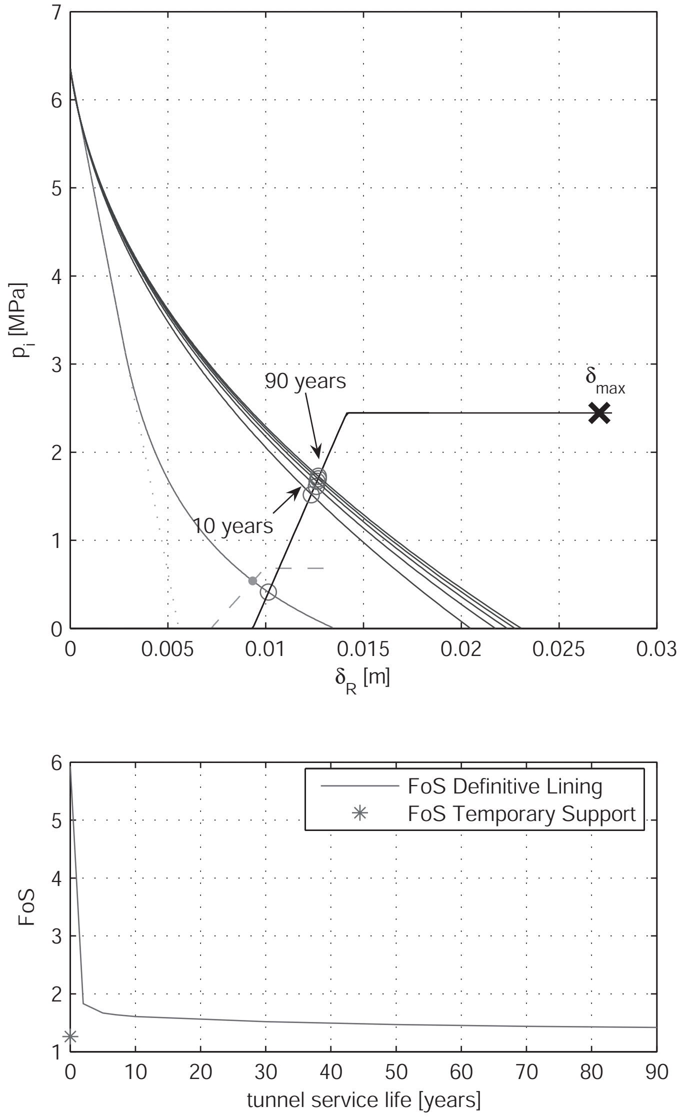


Fig. 6.11. On top: Convergence-confinement analysis with an elastic-viscoplastic rock mass. Parameters after [18]. The GCC is represented after tunnel construction and during tunnel service life each 20 years from 10 years till 90 years. On bottom: FoS evolution with time.

Figure 6.11 shows the evolution of radial wall displacement for the second set of viscoplastic parameters proposed by [18]. By comparing the results with Figure 6.10, it is evident how the equilibrium evolution with time strongly depends on the chosen parameters. Again, a good characterisation of excavated rock mass seems to be necessary for assessing and interpreting the long term conditions. On the bottom of Figure 6.11, the evolution of FoS is represented during tunnel service life. As in the case of viscoelasticity, it is necessary to read this graph with particular caution. Indeed, while the Safety Factor of the temporary support is slightly higher than 1, the initial value of Safety Factor is particularly high (i.e. about 6). It is necessary to observe that:

- The chosen temporary support, common to all cases, and not depending on the excavated rock mass, as it happens in reality, may not be adequate to the excavated rock mass. For example, the use of steel ribs could be found more appropriate for tunnels excavated in marls.
- The high initial value of the Safety Factor for the definitive lining, as already observed, is due to the basic assumption that assimilate the lining behaviour to a circular concrete

ring, under axisymmetric loading conditions, which is rarely respected in real cases. However, the main result remains the significant decrease of the Safety Factor with time.

Weathering

As introduced in Chapter 2, rock mass weathering can be modelled as a long term reduction of mechanical properties (softening). [47] discusses the strength reduction in the broken zone and the resulting changes in support pressure requirements. The effects of weathering agents may change the compressive strength of the rock mass together with its elastic modulus and Poisson's ratio. This progressive loss of strength is essentially due to the cohesion. This approach was also applied by [80] for drawing the long term GCC. Based on the results of several authors [80]; [64] it can be affirmed that on the long term the mechanical properties of a rock mass may reduce of about 30% of their initial value due to chemical weathering (according to [80] the Young's Modulus may reduce up to 40% of its initial value).

The decrease in mechanical properties of a rock mass may also change the long term stability of the tunnel crown ("Roof effect"). This secondary effect should be taken into account as the instability of the zone above the tunnel crown may increase the load that the supporting structure has to bear. The rock mass properties reduction results in a plastic radius increase. Under the effect of gravity forces, the broken zone above the tunnel crown has a tendency to "flow" towards the tunnel (Figure 6.12).

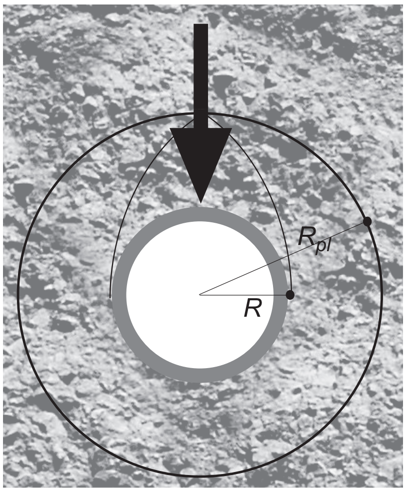


Fig. 6.12. "Roof effect": under the effect of gravity forces, the broken zone above the tunnel crown has a tendency to "flow" towards the tunnel.

The additional pressure on tunnel crown is proportional to the thickness of the broken zone. As explained in Chapter 2, several authors focused on the influence of this zone on the GCC [114]. In the following example, the convergence line in crown has been evaluated by considering the weight of the unconfined zone, as suggested by [111], (see also Equation 2.15, Chapter 2):

$$\Delta p_{crown} = \gamma_{rm} \cdot (R_{pl} - R) \tag{6.14}$$

with γ_{rm} unit weigth of the rock mass above tunnel crown, R tunnel radius and R_{pl} plastic radius. Though this calculation may appear, in some cases, too conservative, the choice has been done in order to represent the most critical situation that may occur.

Using long term parameters it is possible to draw the long term curve vs. the short term one. The influence of weathering on the long term equilibrium of the tunnel is shown in Figure 6.13. The excavated rock mass is a poor quality sandstone (Table 6.2). The strength properties have been reduced of about 30% of their initial value, while the Young's Modulus was kept its initial value. As the weathering rate depends on several parameters, it is quite difficult to find a law for reducing mechanical properties with time. Thus, it has been chosen to simulate only the short and the long term conditions and not their time evolution.

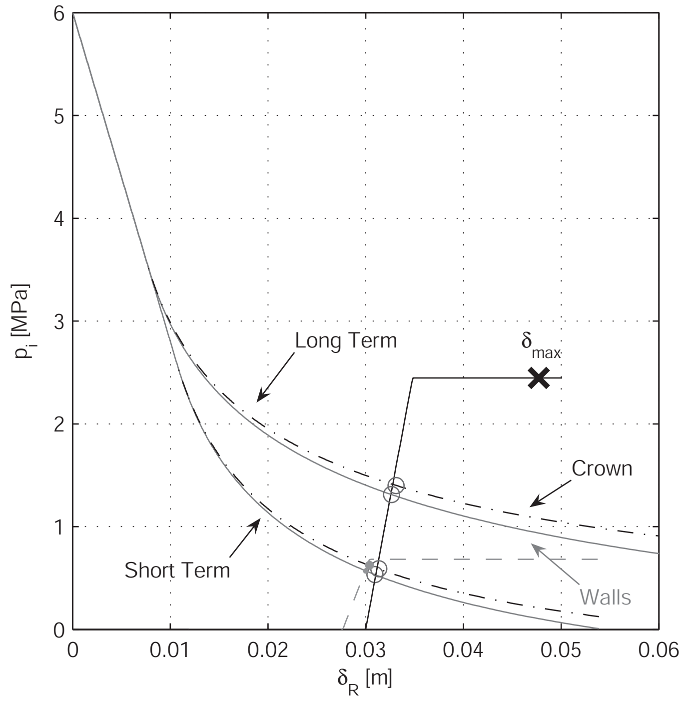


Fig. 6.13. Short and long term equilibrium conditions for a tunnel excavated in poor quality SANDSTONES (see Table 6.2), considering weathering effects on the long term (reduction of about 30% of the short term strength properties) and the roof effects due to loss of confinement of the excavated rock above tunnel crown (under the gravity effect).

As it is possible to see, the load on the temporary support is quite high. This result should be considered with particular caution. As a matter of fact, as it has been previously explained, for sake of simplicity, the same temporary support has been chosen for all types of rock masses, which is far from what happens in reality. For example, in this case, a more adequate support could have been composed of steel ribs and shotcrete.

By comparing short and long term equilibrium it is possible to estimate the evolution of the FoS for the definitive lining. But, in this case, after drawing the GCC in crown and at side walls, an important consideration must be done. As the stress distribution around the cavity does not respect any more the basic assumption of axisymmetric stress field, before estimating the Safety Factor, it is necessary to evaluate the stresses in the lining. The thrust and the bending moment can be evaluated in both crown and walls as reported in [42] (after [93]). Actually, as it is possible to see in Figure 6.13, in both the short and the long terms, the results show that the difference between the equilibrium in crown and side walls is negligible. Thus, the Safety Factor has been evaluated under the hypothesis of axisymmetric loads. Table 6.4 summarises the value for both short and long term conditions respectively in crown and at side walls.

Tab. 6.4. Short and long term FoS values for the definitive lining of a tunnel excavated in poor quality SANDSTONES (see Table 6.2), considering weathering effects on the long term (reduction of about 30% of the short term strength properties) and the roof effects due to loss of confinement of the excavated rock above tunnel crown (under the gravity effect).

	Short Term (Definitive lining)	**Long Term (Definitive lining)**
Crown	4.6	1.9
Side walls	4.1	1.8

A reduction of the strength properties of about 30% of their short term values corresponds to a decrease of the Safety Factor of more than 55% of its initial value (considering only the definitive lining). The difference between the crown and the side walls is reduced in the long term. Of course, as previously observed, this is valid for a perfect circular concrete ring, with an axisymmetric load, when all time effects that may reduce the serviceability of the lining are neglected.

An additional remark should be made regarding the above presented example. As a matter of fact, this example simulates only the strength properties reduction (i.e. c and φ) due to chemical weathering, while the Young's Modulus is kept constant. In reality, as explained in Chapter 2, rock mass weathering causes a mechanical properties reduction. [80], for example, suggested to reduce not only the strength properties but also the Young's Modulus on the long term (i.e. respectively, about 30% of reduction for the strength properties and 40% for the Young's Modulus). Anyway, being a consequence of weathering and not of ageing, this mechanical properties decrease should be applied only in the plastic zone around the tunnel without affecting the elastic one.

Other actions: Consolidation

As most deep tunnels are driven in water saturated rock masses, the re-equilibrium of pore water pressure may change the long term load that the tunnel lining has to bear. As observed by [58] both the hydro-mechanical coupling during excavation and the pore water pressure re-distribution after construction have a significant effect on pressure and displacements. Starting from the solutions developed by Labiouse in 1998 for the undrained response of poro-elasto plastic media around cylindrical cavities, [58] in his PhD thesis, proposed a new design method for taking into account the long term effects of the pore water pressure on tunnel equilibrium. An example of application of this method is proposed in the following for a tunnel excavated in low permeable saturated marls (see Table 6.2, good quality parameters are considered as drained values). The porosity of this formation is 16.5% and the water bulk modulus is $K_w = 2\ \mathrm{GPa}$. The rock mass is assumed as elastic plastic with a Mohr-Coulomb criterion and $\psi = 0°$.

The new equilibrium conditions depend on waterproofing and drainage systems. Considering three different positions of the water table: 10 m, 50 m, 100 m above the tunnel (i.e. pore water pressure, pp_0, equal to 100 kPa, 500 kPa, 1000 kPa), Figure 6.14 shows on a convergence-confinement diagram the long term behaviour for both impermeable and permeable linings.

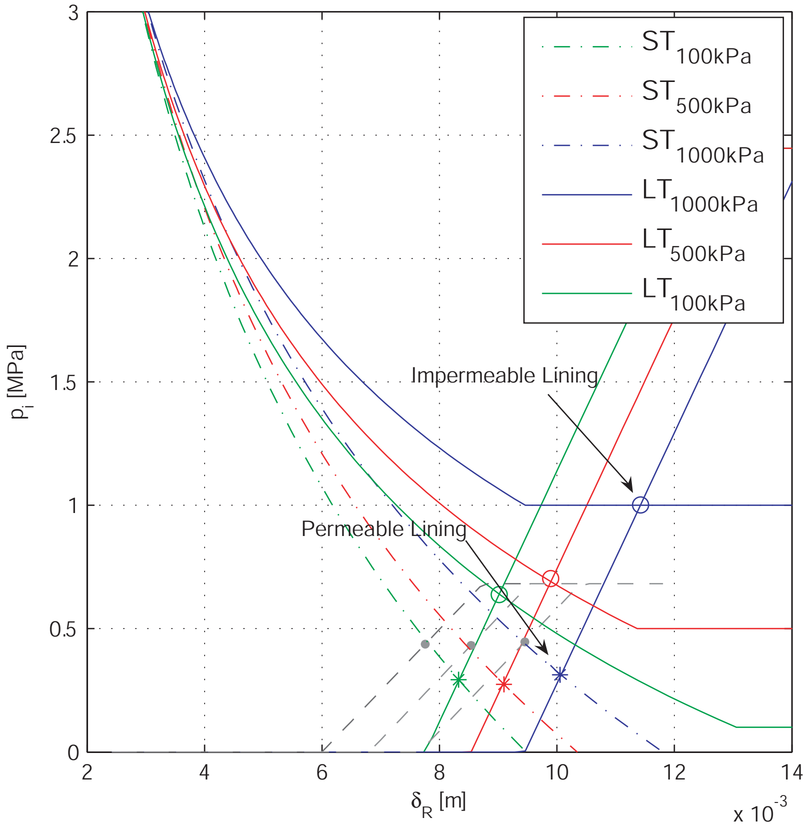


Fig. 6.14. Long term equilibrium conditions for a tunnel excavated in low permeable saturated porous MARLS of good quality (Table 6.2). The GCC is represented for three different initial values of pore water pressure (i.e. green = 100 kPa; red = 500 kPa; blue = 1000kPa). The short term curves (ST) represent undrained conditions of the excavation, while the long term curves (LT) are valid for impermeable linings. The long term equilibrium is calculated for both permeable (i.e. star) and impermeable lining (i.e. circle).

As the rate of the process is a function of rock mass permeability, it has been chosen to evaluate only the changing conditions from short to long term. In the short term the curve represents the rock mass response to a tunnel excavation in undrained conditions; while on the long term it is in drained conditions. The minimum pressure in the long term curve with an impermeable lining is represented by the pore water pressure, pp_0. As it is possible to observe by comparing the two cases, the water boundary conditions (i.e. permeable or impermeable lining) strongly influence the long term equilibrium of the tunnel. In case of impermeable lining the increase in pressure and displacements is remarkable if compared to permeable structures.

Assuming, as before, that the definitive lining is not affected by degradation with time, the Safety Factor on the long term reduces only in case of an impermeable lining. The reduction is more significant in case of a high initial pore water pressure in the rock mass (pp_0). Table 6.5 summarises the values of the definitive lining for both short and long terms and considering an impermeable lining.

Tab. 6.5. Tunnel excavated in low permeable saturated MARLS of good quality (Table 6.2). Evolution of the Factor of Safety (FoS) of the definitive lining due to the re-distribution of pore water pressures in case of an impermeable lining.

pp_0 (kPa)	Short Term (Definitive lining)	Long Term (Impermeable)
100	8.4	3.8
500	8.7	3.4
1000	7.4	2.4

6.3.2 Long term confinement line

During tunnel excavation only the support assures the equilibrium of the tunnel. Then, after the concreting of the definitive lining, the support is gradually affected by degradation and the lining starts to contribute to the tunnel stability. Thus, when the support is completely degraded only the lining assures the equilibrium of the cavity. This process changes the equilibrium conditions as shown in Figure 6.15. On top are represented the changing conditions:

1. temporary support only (placed at 2 m of distance from the tunnel face),
2. temporary support + definitive lining just after its construction,
3. 50% degraded temporary support (i.e. $k_s = 50\% \cdot k_{s0}$ and $p_{max} = 50\% \cdot p_{max0}$) + definitive lining,
4. definitive lining only, i.e. when the temporary support is fully degraded.

On bottom the FoS evolution for progressive deterioration of temporary support. On the short term, during tunnel construction the Safety Factor is evaluated by considering only the support system. When the lining is placed the Safety Factor increases as both the temporary support and the definitive lining (in an ideal case) contribute to tunnel equilibrium. With time passing the equilibrium evolves as a function of support degradation. As both the support pressure and the equilibrium pressure decrease, it is possible to observe that the system reach a minimum value of Safety Factor (i.e. $k_s(t) = 30\% \cdot k_{s0}$). After that, the Safety Factor slightly increases up to the final value that corresponds to the equilibrium between the definitive lining and the rock mass. As already observed in other examples, the Safety Factor results in quite high values. This is due to the basic assumption of a perfect axisymmetric load on a perfect circular concrete ring. In reality, these conditions are rarely met and it is reasonable to assume that the Safety Factor values are lower. Nevertheless, it is interesting to observe how the Safety Factor changes by considering that the definitive lining and the temporary support are working together.

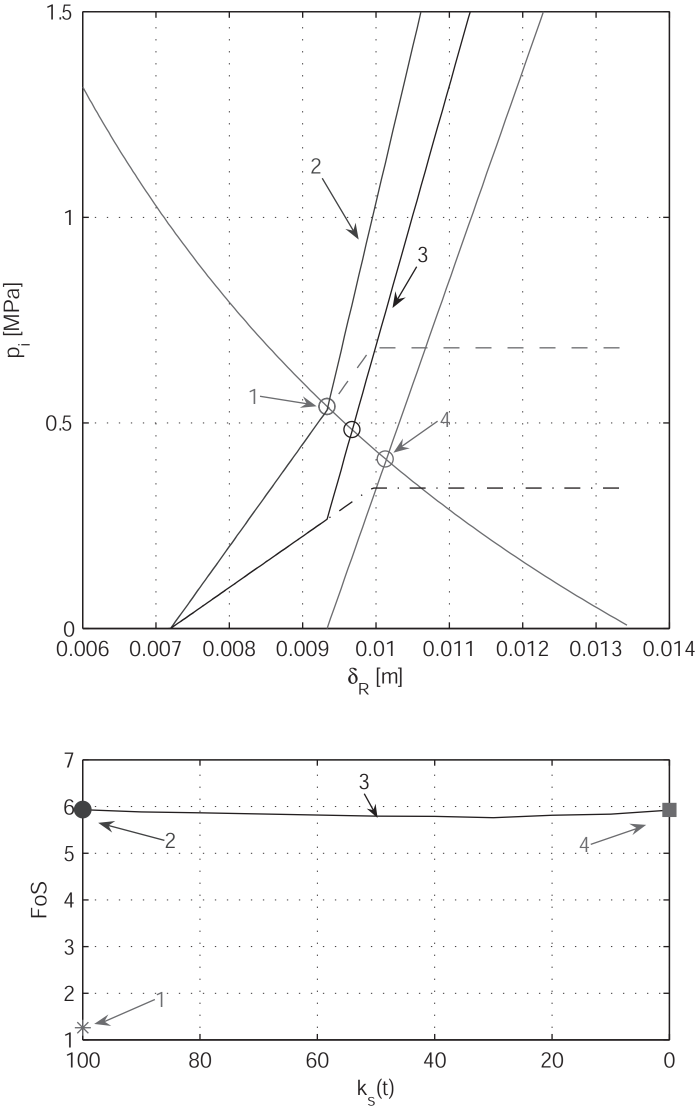


Fig. 6.15. On top: progressive changing of loading conditions with degradation of support with time for a tunnel excavated in good quality MARLS (see Table 6.2). On bottom: FoS evolution with degradation of support with time. The Safety Factor is calculated by considering together definitive lining and temporary support.

As during tunnel service life the definitive lining may be affected by several disorders (see Chapter 5). This degradation corresponds to a properties decrease (i.e. strength and stiffness), thus, on the long term, it should be necessary to take into account also these effects. As introduced in Chapter 2, both ageing and weathering may modify tunnel lining conditions with time. In particular:

- ageing can be modelled by increasing strain,
- weathering can be modelled by decreasing lining mechanical properties or thickness.

Ageing

As already observed by [94], about 75% of the long term creep of concrete occurs in only 1 year. As explained previously, creep depends on the rate of strain (and displacements) which is a function of the ratio between the applied load and the strength of the material. Moreover, as shown in Figure 6.15 it may take some time before the lining starts to be loaded. For all these reasons, the creep effects are considered negligible for evaluating the long term conditions of the concrete lining.

Weathering

The degradation rate of the temporary support may change a lot from tunnel to tunnel mainly due to constructive features. In particular, as demonstrated in Chapter 5 the waterproofing system position, with respect to both support and lining, may influence a lot the degradation speed.

Based on both the consideration reported in Chapter 2, and the results of the analyses conducted on the tunnel data base (Chapter 5), it is possible to identify two types of weathering processes that characterise the concrete of the definitive lining:

I. When the waterproofing system is absent or doesn't work properly, the definitive lining is exposed to external attacks at both the intrados and the extrados. This process causes mechanical properties reduction due to possible leaching of aggressive ground water from the extrados to the intrados of the lining (e.g. sulphates corrosion, calcium leaching...).

II. When a waterproofing membrane is placed at the definitive lining extrados, under normal condition, the only active surface in terms of degradation is the lining intrados. This process causes lining thickness reduction and it is mainly caused by de-icing salts corrosion.

Both processes may result in changing the long term behaviour of the lining structure. An example of mechanical properties reduction on tunnel equilibrium (i.e. case I) is shown in Figure 6.16, for a tunnel excavated in poor quality sandstones (Table 6.2). As shown by [25]; [96] (see Equation 2.7 to Equation 2.10, Chapter 2), the reduction of mechanical properties (i.e. both the Young's Modulus and the strength) due to concrete weathering (e.g. caused by Calcium leaching) is considered proportional to the degraded area (i.e. thickness):

$$X_d = a \cdot \sqrt{t} \tag{6.15}$$

by choosing $a = 5.2\mathrm{E}\text{-}4 \ \frac{\mathrm{m}}{\mathrm{days}^{0.5}}$.

This rate has been estimated starting from laboratory tests conducted on concrete lining specimens after about 30 years of service life of the Flonzaley tunnel (Vaud Canton) as shown in Chapter 7 (Section 7.3). Though, the mechanical properties of a normal concrete improve with time, at least at the beginning of the tunnel service life, it can be considered that degradation rate is predominant. The corresponding evolution of Safety Factor with time is also represented (Figure 6.16, on bottom). Once again, the high Factor of Safety values are related to the basic assumption of the Convergence Confinement method (axial-symmetry) and should be considered as rarely representative of reality. Nevertheless, from a qualitative point of view, it is important to note that the Safety Factor reduction due to lining weathering is less significant than the one observed in case of rock mass degradation.

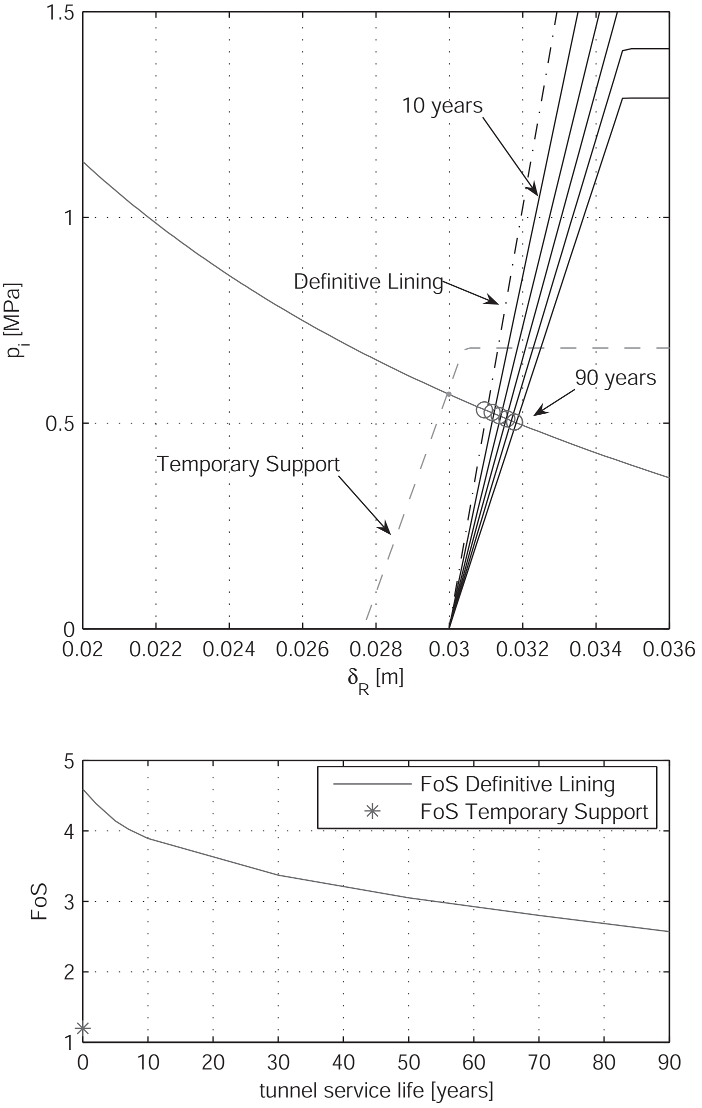


Fig. 6.16. Convergence-confinement analysis for a tunnel without waterproofing system excavated in a poor quality SANDSTONE (see Table 6.2). External attack due to aggressive ground water leaching. On top: weathering of the definitive lining. On bottom: FoS evolution with time.

For what concerns case II, some additional considerations must be done. As in any other concrete structure, in a tunnel lining it is possible to recognise the concrete surface (“skin”), characterised by a lower quality, and the core of the structure with a higher quality. As introduced in Chapter 2, in case of plain concrete the main problem is represented by lining degradation due to frost de-icing salts attack. As shown in Figure 6.17, for a plain concrete lining, two different evolution rates may be considered.

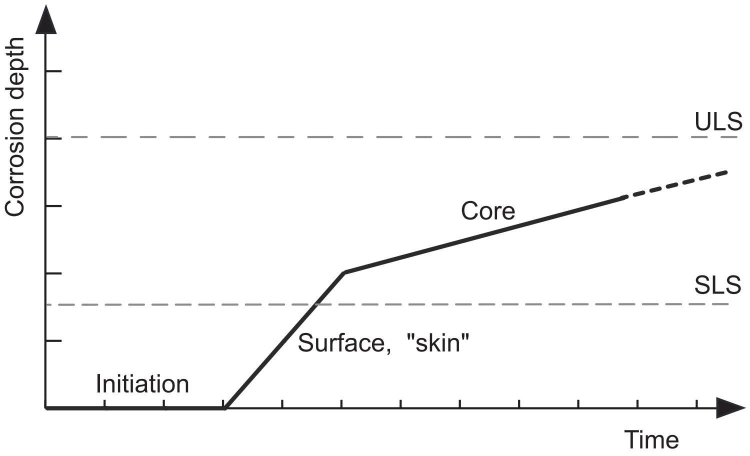


Fig. 6.17. Plain concrete lining weathering by de-icing salts attack: process initiation and evolution rate (after [22]; [41]).

Due to its poorer quality the concrete lining surface is thus subjected more easily to de-icing salts penetration and attack. Thus, if a regular maintenance is performed during tunnel service life, it can be assumed that only the surface of the lining is really affected by scaling due to de-icing salts frost attack. The process initiation and effects are influenced by humidity, temperature and by salt concentration (i.e. according to [74], pessimal conditions are represented by: $NaCl$ concentration of about 2-4% with homogeneously saturated porous material that correspond to the lower part of the side walls and gutter).

As reported in [74]; [22]; [129] after several applications of de-icing salts, the maximum Chlorides content is found at a certain depth from the concrete surface (e.g. about 40-50 mm). According to the freezing point depression by the salt solution, the concrete structure can be divided into different layers which would freeze at different temperatures. This may originate unexpected tensile stresses in the structure and causes concrete surface scaling. Depending on the exposure class the affected surface may be more or less thick. Unfortunately, there is still not enough information about the rate of this process in tunnels and only some approximation can be done. Thus, the rate of the process has been chosen based on considerations about the exposure classes (see Chapter 2) together with inspection results (i.e. Vaud and Ticino Cantons in particular).

Considering a degraded thickness proportional to the square root of time (Equation 6.15), two examples, representing two different exposure classes, (i.e. respectively splash and mist) are shown in the following, for a tunnel excavated in good quality marls (Table 6.2):

- For the splash exposure (Figure 6.18, left, mainly representing what happens to the lower part of the side walls) it has been chosen $a = 5\text{E-}4\ \frac{\text{m}}{\text{days}^{0.5}}$ that corresponds to a thickness reduction of about 50 mm after 30 years of tunnel service life.
- For the mist exposure (Figure 6.18, right, mainly representing what happens to the higher part of the side walls and tunnel crown or, in case, to the ventilation slab), it has been chosen $a = 2.5\text{E-}4\ \frac{\text{m}}{\text{days}^{0.5}}$ that corresponds to a thickness reduction of about 25 mm after 30 years of tunnel service life.

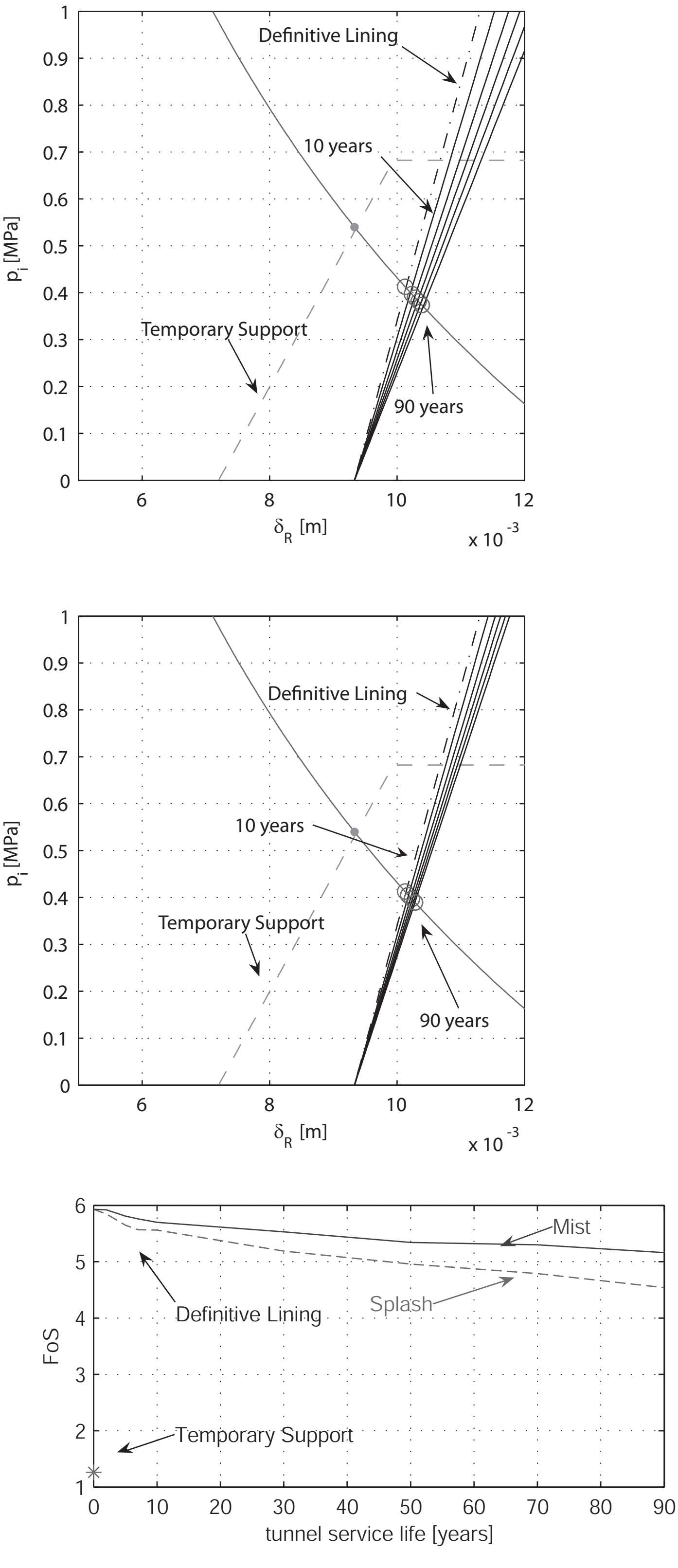


Fig. 6.18. Convergence-confinement analysis for a tunnel with waterproofing system excavated in good quality MARLS (see Table 6.2). Frost de-icing salts attack at tunnel intrados. On top: weathering degradation of the definitive lining. On the left: splash exposure (XF4), on the right: mist exposure (XF2), as described in Chapter 2. On bottom: FoS evolution with time.

As it is possible to observe, in both cases, the FoS evolution with time for the definitive lining is less affected by this kind of degradation if compared to previous values obtained for ground water chemical aggression (Figure 6.16). High values are still the consequence of the Cv-Cf basic assumption and the results should be considered only from a qualitative point of view. The most interesting result is that lining deterioration seems to have negligible effects on long term tunnel stability if compared to rock mass degradation. Anyway, it is important to keep in mind that this example is an important simplification of the problem as the concrete characteristics may change a lot due to its composition. For example, as already mentioned in Chapter 2, the special cements with very low content of C_3A (i.e. less than 3%), prescribed in case of potential sulphates attack, may show low resistance to Chlorides attack and could show major problems that should be further investigated. Also casting conditions (e.g. concrete vibration) may affect the resulting Safety Factor. For example, it may increase the concrete porosity and thus accelerate the degradation rate. Moreover, also maintenance frequency and type may strongly influence the rate of this process.

In case of reinforced concrete structures (e.g. tunnel portals, ventilation slabs, side walls foundations and, occasionally, whole supporting structure), the situation may completely change. The main problem, in this case, is represented by Chlorides penetration with consequent corrosion of steel bars (see Chapter 2). Thus, in case of reinforced concrete lining, for assessing the stability conditions of the structure, it may be quite important to know the penetration depth and the free Chlorides content below the lining surface. As shown for a real case (i.e. Flonzaley Tunnel, Vaud Canton, see Chapter 7), Chlorides profiles allows to determine the amount of free Chlorides at a certain depth from the concrete surface. Once the corrosion of steel bars is activated, depending on tunnel conditions (e.g. aggressive atmosphere, humidity, reinforcement type, and presence of fissures and cracks on the lining surface), the process may be more or less fast. The concrete spalling, due to steel corrosion, results in a lining thickness decrease and thus the lining bearing capacity may be strongly affected. In particular, these effects should be taken into account for assessing stability conditions of tunnel portals, usually constructed with reinforced concrete.

6.3.3 Long term Convergence & Confinement lines

As the changing conditions of both the rock mass and the lining influence the tunnel stability conditions on the long term, it is necessary to describe the Safety Factor evolution considering that the degradation processes affect both the rock mass and the lining.

Tunnel weathering

If a tunnel is excavated in a sedimentary rock mass, as demonstrated in Chapter 5, on the long term it may show disorders due to the rock mass weathering potential. Moreover, when the waterproofing system is absent or doesn't work properly, the concrete lining may be affected by long term weathering due to the aggressive ground-water action, as described in Section 6.3.2. Thus, in this case, both the convergence and the confinement lines change on the long term together with equilibrium conditions.

Weathering of both rock mass and lining has been modelled as follows:

- For what concerns the rock mass a poor quality sandstone (Table 6.2) has been considered and strength properties reduce with time according to an hyperbolic law:

$$V(t) = V_{ST} - \left[(V_{ST} - V_{LT}) \cdot \left(1 - \frac{1}{1 + \frac{t}{T}} \right) \right] \tag{6.16}$$

where V_{ST} is the short term value of the parameter (i.e. friction angle φ and cohesion c), V_{LT} is the value on the long term of the same parameter, considering a reduction of 30% of its initial value (V_{ST}) as suggested by [80] and commonly used in Rock Mechanics for long term strength parameters (see Chapter 2), t is time (in years) passed since the tunnel construction (i.e. tunnel service life), and T is a constant that defines the rate of the rock mass weathering process and which has been considered, in this example, equal to 1.

- For what concerns the concrete lining, mechanical properties reduction due to aggressive ground water leaching has been considered proportional to the degraded area, as described in Section 6.3.2 (see Equation 6.15), with a weathering rate equal to $a = 5.2\text{E-}4 \dfrac{\text{m}}{\text{days}^{0.5}}$.

Figure 6.19 on top shows the evolution of the equilibrium point considering both rock mass and lining weathering. As it is possible to see, due to the weathering process an additional load is considered on tunnel crown and two GCC are represented respectively for side walls (i.e. continuous curves) and crown (i.e. dotted curves). Due to this loss of confinement at the tunnel crown the axisymmetric condition is no longer satisfied and, for a correct assessment of the tunnel stability, the stress in the lining should be evaluated. Anyway, as already observed in Section 6.3.1, both in short and long term conditions the difference between the two curves is negligible (i.e. about 100 kPa), thus the Safety Factor has been evaluated under the hypothesis of axisymmetric loads respectively for crown and side walls. By comparing the short term and the long term conditions it is possible to observe an important reduction in the Factor of Safety (Figure 6.19 on bottom) already after 10 years of service life, mainly due to the loading conditions increase caused by the rock mass weathering. Then, the reduction is slower, mainly depending on the concrete lining weathering. Moreover, according to the simulation results a failure could be expected before the end of the service life.

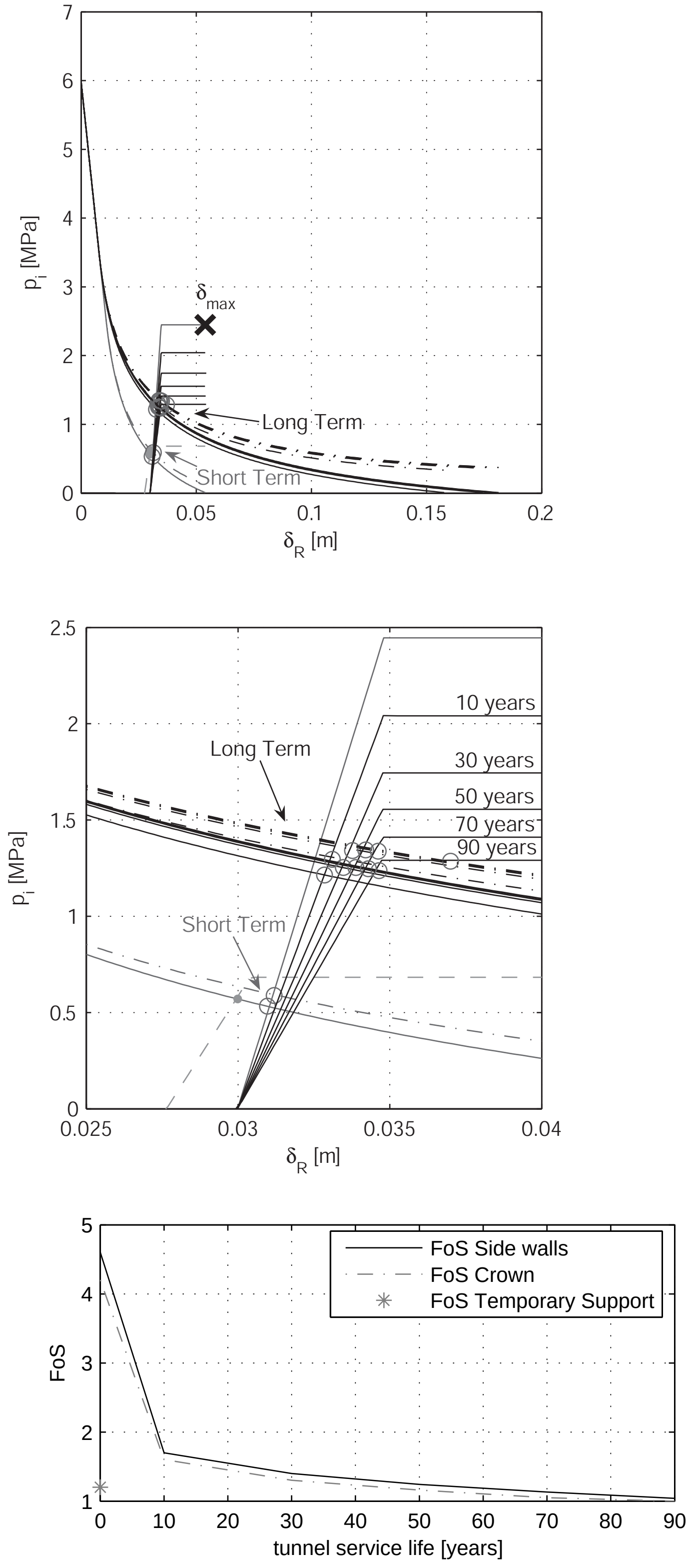


Fig. 6.19. On top: short and long term equilibrium conditions of a tunnel without waterproofing system excavated in poor quality SANDSTONES (see Table 6.2), considering weathering effects on both the rock mass (i.e. strength properties reduction up to 30% of their initial values) and the concrete lining (i.e. mechanical properties reduction due to aggressive ground water leaching). The GCC in crown (dotted curve) takes into account the roof effects (i.e. the loss of confinement of the excavated rock above tunnel crown, under the gravity effect, due to rock mass weathering). On bottom: FoS evolution with time considering the GCC, respectively at tunnel crown et side walls, under axisymmetric load.

The consequences on the long term are worse by considering the deterioration (and weakening) of both the rock mass and the lining, as it is possible to observe by comparing these results to the previous ones (i.e. Section 6.3.1 and Section 6.3.2). A good estimation of the rock mass degradation rate is necessary since it has a great influence on the evolution of the Safety Factor of the tunnel. Actually, by changing the value of the constant T in Equation 6.16 (i.e. by reducing the rate of the rock mass weathering process) the reduction of the Safety Factor may depend more on the concrete lining degradation.

6.4 Conclusion

A specific methodology for evaluating tunnel conditions during its service life has been presented. By means of this methodology, a global update of the input parameters used for analysis and prediction, allows to take into consideration the long term effects on the global condition of the tunnel. The original input data (e.g. rock mass properties and behaviour; tunnel geometry, construction method; supporting structure characteristics and behaviour) used for evaluating tunnel initial condition, may change with time passing. Thus, after each principal inspection, it is necessary to estimate degradation effects and to model them in terms of changing properties. New equilibrium conditions can be, then, evaluated by using up-to-date input to the problem.

Starting from the classical solutions for a deep circular tunnel excavation (convergence-confinement method), some examples of tunnel degradation have been presented, taking into account the long term behaviour of both tunnel lining and excavated rock mass. In this way, it has been possible to show how operation and environmental conditions that characterise tunnel service life may be considered in long term tunnel equilibrium assessment.

The contributions of the rock mass and the lining have been analysed in a separate way. Then, by observing the change of the intersection point between the two curves at each time t, their interaction has been evaluated in terms of Factor of Safety FoS evolution during tunnel service life. Though simplistic, the proposed approach gives a chance for first estimating the effects of influence factors on tunnel equilibrium conditions. Moreover, though the basic assumptions are rarely met in real conditions, it helps in pointing out some important observations:

1. In order to verify the long term equilibrium of a tunnel, it is necessary to take into consideration the delayed behaviour of both excavated rock mass and supporting structure.
2. By representing through simplified models the degradation processes, it is possible to illustrate modifications in tunnel equilibrium conditions.
3. The delayed behaviour of the rock mass, compared to lining degradations, has stronger effects on changing the long term stability conditions. For example, changes in loading conditions in case of viscous (i.e. both viscoelastic and viscoplastic rock masses) may strongly affect the long term Safety Factor while lining weathering produces smaller effects.
4. In order to avoid major problems, appropriate rheological models for describing the rock mass long term behaviour, should be chosen already at the tunnel conception and design steps.

Anyway, the basic assumptions of the convergence-confinement method show how this tool is not very adequate to represent the real situation of a degraded tunnel. The local character of several pathologies could not be taken into consideration. The perfect axial-symmetric condition assumed by the Cv-Cf method induces an overestimation of the Factor of Safety, as observed in several examples. As a matter of fact, the axial symmetry of loads together with the maximal load that a perfect circular ring of concrete in good quality may bear, are rarely representative of reality, i.e.:

1. As observed for the majority of the Swiss National Roads tunnels, in very few cases tunnel has a perfect circular shape.

2. The loading conditions, except for very deep tunnels, do not respect the axial-symmetric assumption.

Finally, the origin of the confinement line, which represents the radial displacement the tunnel has wall already undergone when the support is placed, has clearly a major influence on the final equilibrium state. Consequently, all information about construction details should be taken into account for a correct long term stability assessment.

However, the methodology described at the beginning of this chapter, together with the degradation models used for evaluating long term conditions of both rock mass and lining (and described in Chapter 2), may be adopted also with long term stability evaluation by means of other calculation methods such as more refined numerical analysis (FEM) or the reactions method.

7. Case Studies

Examples of interpretation and analysis of delayed phenomena are described in this chapter for three tunnels of the Swiss National Roads:

1. The first example proposes a simple method for the interpretation of monitoring measurements (Mont Russelin Tunnel, A16 - Jura Canton).
2. The second analyses the long term stability conditions of the Mont Terri Tunnel (A16 - Jura Canton) considering the delayed behaviour of the rock mass (i.e. viscoplastic model, see also Chapter 6).
3. The third example describes two simple methods for simulating the effects of concrete lining weathering and evaluating its consequences in terms of mechanical and geometrical characteristics of the supporting structure (Flonzaley Tunnel, A9 - Vaud Canton).

7.1 Monitoring Measurements Interpretation - Mont Russelin Tunnel

When a tunnel is excavated in critical geological conditions, especially for what concerns recent tunnels, a monitoring system can be installed for controlling the stress-strain evolution with time. As a matter of facts, stress and strain monitoring may be very useful in predicting the long term behaviour of a tunnel. In the following it is shown how it is possible to verify the stability conditions of a tunnel after several years of operation, using monitoring data.

As observed in Chapter 5, tunnels excavated in the Jura Mountains may show symptoms of rock mass ageing. The rock mass delayed behaviour causes side walls displacements and changes the tunnel shape. The following paragraph explains how tunnel lining stability conditions can be assessed from strain measurements. The method is based on a simplified approach, described by [42] (after [93]), for evaluating the lining stresses caused by tunnel ovalisation (i.e. from circular to egg shaped).

7.1.1 Mont Russelin tunnel (JU)

The Mont Russelin tunnel belongs to the A16 axis in the Jura Canton. Particular attention was given to geological conditions as the tunnel crosses marls and clays formations that may show manifest symptoms of delayed behaviour. Moreover, the tunnel crosses the Keuper formations, where it was expected to observe complex phenomena due to swelling clays together with chemical reaction between anhydrite and water. After the important problems encountered in the same rock formations during the construction of the Belchen tunnel some years before [78], particular attention has been paid when choosing excavation section and construction methods. Moreover, as laboratory tests conducted by LMR-EPFL had pointed out a potential critical delayed behaviour of the excavated ground masses, a monitoring system was installed for controlling stresses and strains evolution at tunnel contour. The excavation began in 1989. The tunnel, 3560 m in length, has a circular cross section of about 110 m^2 (i.e. equivalent radius, R = 6 m) and has been excavated with a full face shield, avoiding the presence of water. Precast concrete elements of about 26 cm thickness were used as primary support. The definitive lining is a cast in place concrete ring of about 34 cm. The maximum overburden, more than 400 m, is attained in the central part where the tunnel crosses the marls.

7.1.2 Monitoring results

Stresses and strains evolution has been monitored in 8 tunnel sections since the tunnel construction finished, in 1991, and during operation until the year 2000. The monitoring sections are equipped as follows:

- four extensometers (Interfels GmbH), at TM 1276.25, TM 1906.50, TM 2253.50, TM 2533.25, placed as shown in Figure 7.1;
- four pressure cells (Glötzel GmbH), at TM 1262.50, TM 1939, TM 2304.75, TM 2520.75, placed between the temporary support and the lining.

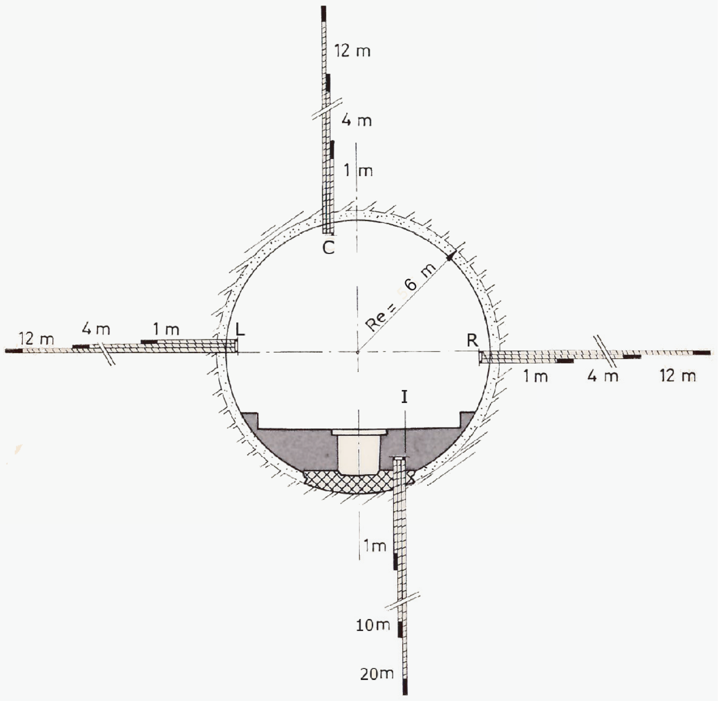


Fig. 7.1. Typical monitoring section equipped with 4 extensometers (after [CMR - Consortium d'Etude Mont Russelin, 1990]).

Table 7.1 shows, for each section, the evolution of radial displacements measured by the four extensometers after the definitive lining concreting, respectively, in crown $\delta_{crown} = \delta_1$, invert $\delta_{invert} = \delta_2$, left $\delta_{left} = \delta_3$ and right $\delta_{right} = \delta_4$ side walls. After calculating the mean displacement, δ_m (Equation 7.1), these measurements have been transformed into a lateral displacement, δ_0 (Equation 7.2), as shown in Figure 7.2.

$$\delta_m = \frac{\delta_1 + \delta_2 + \delta_3 + \delta_4}{4} \tag{7.1}$$

$$\delta_0 = \frac{\delta_3 + \delta_4}{2} - \delta_m = \frac{\delta_3 + \delta_4 - \delta_1 - \delta_2}{4} \tag{7.2}$$

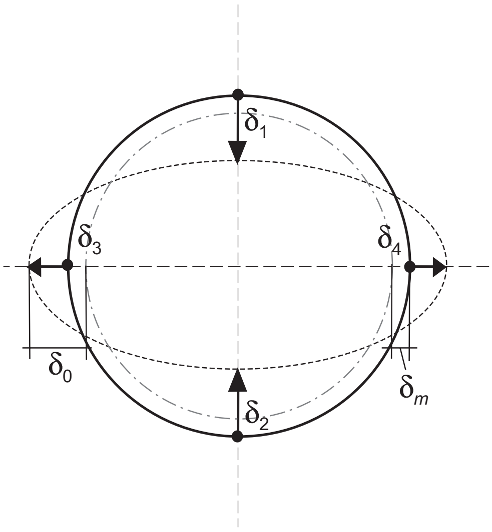


Fig. 7.2. Tunnel ovalisation: mean displacement δ_m *and lateral displacement* δ_0*.*

Thus, the lateral displacement corresponds to $\delta_0 = \frac{\delta_h - \delta_v}{2}$, with, respectively, δ_h mean of horizontal and δ_v mean of vertical displacements.

Tab. 7.1. Measurements, mean δ_m *and lateral displacements* δ_0 *(mm) for each monitored section. (+) stands for inward displacement and (-) for outward displacement. Note that, though values are given with two decimals, the precision of an extensometer is* $1/10$ *of mm.*

Section	$\delta_{crown} = \delta_1$	$\delta_{invert} = \delta_2$	$\delta_{left} = \delta_3$	$\delta_{right} = \delta_4$	δ_m	δ_0
1	2.38	2.19	-0.26	-0.1	1.05	-1.23
2	--[a]	0.75	-0.17	-0.15	--	--
3	1.15	-0.11	0.53	-0.05	0.38	0.14
4	2.83	-0.38	-0.07	0.17	0.64	-0.58

a.The extensometer was broken.

Interpretation

If the loading condition is not axisymmetric, the tunnel section deforms from circular to egg shaped. In addiction to the thrust, a bending moment originates thus in the lining. When the induced stresses exceed the material strength lining failure may occur.

As reported in [42], [93] evaluates the effects of these changes of tunnel shape by considering an ovalisation as shown in sketch of Figure 7.3:

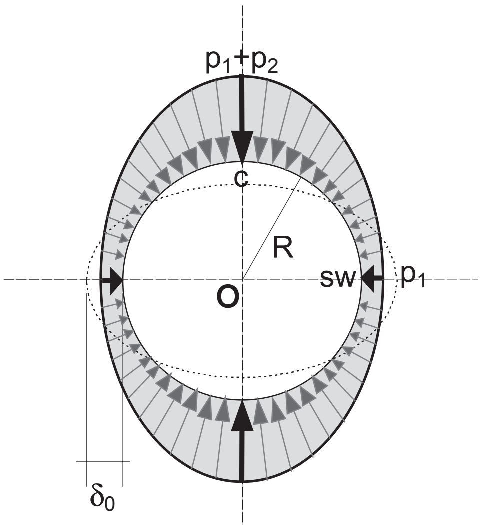


Fig. 7.3. Tunnel ovalisation due to asymmetric loading conditions. δ_0 *(-) if outwards.*

The rock mass reaction, p_{lat}, due to the tunnel lateral displacement, δ_0, is assumed elastic:

$$p_{lat} = -\frac{E}{1+\nu} \cdot \frac{\delta_0}{R} \tag{7.3}$$

Knowing the external loads, p_1 and p_2, it is possible to evaluate the bending moment in the tunnel lining, respectively at side wall (M_{sw}) and crown (M_c):

$$M_{sw} = -M_c = \left(\frac{p_2}{6} + \left(\frac{E}{1+\nu} \cdot \frac{\delta_0}{R}\right)\right) \cdot R^2 \tag{7.4}$$

with

- E Young's Modulus of the rock mass, which changes for each class of excavated rock,
- ν Poisson's ratio of the rock mass, which changes for each class of excavated rock,
- $R = 6$ m, tunnel radius,
- δ_0 tunnel lateral displacement, that changes for each monitored section.

The thrust can be derived from the tangential stress at side wall, N_{sw}, and crown, N_c (note that compression is considered as positive):

$$N_{sw} = \left(p_1 + \frac{2 \cdot p_2}{3} + \left(\frac{E}{1+\nu} \cdot \frac{\delta_0}{R}\right)\right) \cdot R \tag{7.5}$$

$$N_c = \left(p_1 + \frac{p_2}{3} - \left(\frac{E}{1+\nu} \cdot \frac{\delta_0}{R}\right)\right) \cdot R \tag{7.6}$$

Due to the symmetrical conditions the shear force is zero in those two points.

Due to the tunnel ovalisation, the lining curvature changes, originating a bending moment, M, that reaches its maximum value at side walls and crown (i.e. respectively, points "sw" and "c" in Figure 7.3):

$$M_{sw} = -M_c = -\left(E_c \cdot I \cdot \frac{3 \cdot \delta_0}{R^2}\right) \tag{7.7}$$

with

- $E_c = 20$ GPa, Young's Modulus of the concrete lining, $\nu_c = 0.2$ Poisson's ratio of the concrete lining
- $s = 0.34$ m, concrete lining thickness,
- $I = \frac{s^3}{12}$, inertia of the lining section.

The loading conditions can be obtained as follows:

1. p_2 by equating Equation 7.7 and Equation 7.4:

$$p_2 = -\frac{6}{R^2} \cdot \delta_0 \cdot \left(\frac{E_c \cdot s^3}{4 \cdot R^2} + \frac{E \cdot R}{1+\nu}\right) \tag{7.8}$$

2. p_1 knowing the displacement δ_m of the concrete ring caused by axisymmmetric pressure distribution:

$$p_1 = \frac{\delta_m \cdot \frac{E_c}{1-\nu_c^2} \cdot s}{R^2} \tag{7.9}$$

   where $\delta_m = \frac{\delta_v + \delta_h}{2}$ with, respectively, δ_v mean of vertical and δ_h mean of horizontal displacements.

   For each monitored section, the pressures calculated from the measured displacements have been compared to the pressures directly measured from Glötzel cells located at the crown, invert and side walls.

3. The vertical and horizontal pressures, respectively, p_v and p_h, by adding to the external loads, p_1 and p_2, the lateral reaction of the rock mass plat due to the tunnel ovalisation:

$$p_v = p_1 + p_2 + \left(\frac{E}{1+\nu} \cdot \frac{\delta_0}{R}\right) \tag{7.10}$$

$$p_h = p_1 - \left(\frac{E}{1+\nu} \cdot \frac{\delta_0}{R}\right) \tag{7.11}$$

Table 7.2 shows for each section:

- Young's Modulus E and Poisson's ratio ν of the rock mass, which have been estimated from laboratory tests conducted on cored specimens collected during the geological survey [BGA J. Norbert & B. Schindler, 1988],
- the lateral displacement δ_0 (Equation 7.2),
- the pressures p_v and p_h calculated from the measured displacements,
- the mean vertical pressure $p_{v,\,meas}$, and the mean horizontal pressure $p_{h,\,meas}$ evaluated from the pressure cells measurements after 9 years since tunnel lining construction.

Tab. 7.2. Calculation data and results vs. pressure cells measurements

Section	E [a] **(MPa)**	ν **(-)**	δ_0 **(mm)**	p_v **(MPa)**	$p_{v,meas}$ **(MPa)**	p_h **(MPa)**	$p_{h,meas}$ **(MPa)**
1	260	0.3	-1.23	0.5	0.10	0.17	0.07
3	230	0.3	0.14[b]	0.07	0.02	0.10	0.05
4	130	0.3	-0.59	0.2	0.02	0.08	0.08

a.These values are the results of laboratory tests [BGA J. Norbert & B. Schindler, 1988].

b.A positive value means that the ovalisation of the section is in the vertical sense, thus the higher pressure is at the side walls.

As it is possible to see, (Table 7.2), the agreement is satisfactory, considering that this calculation is affected by the precision of the displacements measurements. Nevertheless, it is necessary to observe that the values measured by the pressure cells are strongly influenced by their position. As the cell pressure is placed at the lining extrados, if some gap exists between the support and the lining, the cell pressure may underestimate the loading conditions. Moreover, the Young's Modulus of the concrete lining comes from construction details and may be affected by some error. For what concerns the Young's Modulus of the rock mass in the different sections, these values have been estimated from laboratory tests conducted on specimens collected during the geological survey in a nearby tunnel (the Mont Terri tunnel), which crosses similar rock formations.

Finally, by evaluating thrust and bending moment respectively at tunnel crown and side walls, it is possible to verify if the lining can bear the measured pressures (Table 7.3).

Tab. 7.3. Thrust N and bending moment M at tunnel crown c and walls sw, and respective stresses σ at lining intrados (I) and extrados (E).

Section	p_1 **(MPa)**	p_2 **(MPa)**	δ_0 **(mm)**	N_c **(MN)**	N_{sw} **(MN)**	$M_{sw} = -M_c$ **(MN · m)**	$\sigma_{c,I}$ **(MPa)**	$\sigma_{c,E}$ **(MPa)**	$\sigma_{sw,I}$ **(MPa)**	$\sigma_{sw,E}$ **(MPa)**
1	0.03	0.11	-1.23	0.7	0.4	0.007	2.3	1.6	1.5	0.8
3[a]	0.01	0.09	0.14	0.2	0.5	-0.0008	0.6	0.7	1.5	1.6
4	0.01	-0.04	-0.59	0.9	-0.2	0.003	2.9	2.6	-0.3	-0.6

a.The ovalisation is in the vertical sense, thus p_1 is at crown and $p_1 + p_2$ at side walls.

As it is possible to see, only very small tensile stresses originate in the lining for the third section at side walls. The compressive stresses are very low if compared to the maximum strength of the concrete (i.e. $f_c = 40$ MPa) and the tunnel stability is assured.

7.1.3 Conclusions

Changes in tunnel shape can be described using, for example, extensometer measurements. Tunnel ovalisation (i.e. from circular to egg shaped) due to asymmetric loading conditions causes bending moment (M) originating in the tunnel lining. In this example, assuming an elastic reaction of the rock mass to the tunnel lateral displacement [42], bending moments and axial loads (i.e. thrust N) and the respective stresses originating in the lining have been calculated and compared with pressure cells measurements.

Considering the monitored sections, the comparison between calculated and measured values is acceptable. The differences between measurements and calculation results could be explained by the uncertainties that characterise the measurements and by the fact that the geological parameters have been estimated from laboratory tests on specimens cored during the excavation of the Mont Terri tunnel which crosses similar rock formations. This example shows the possibility of interpreting displacements monitoring results through a simple model for estimating the stresses in the tunnel lining.

7.2 Long Term Ageing of the Rock Mass - Mont Terri Tunnel

When a tunnel crosses rock masses with potential delayed behaviour it may show additional strain on the long term. Depending on the rock mass type it is possible to define a specific rheological behaviour for estimating the long term conditions of the tunnel. Short and long term conditions of the Mont Terri tunnel (Jura Canton, Switzerland) have been estimated with the Convergence-Confinement method. Based on the solution proposed by [17] rock mass ageing has been modelled with a viscoplastic behaviour according to Lemaitre's creep law.

7.2.1 Mont Terri tunnel

The Mont Terri tunnel belongs to the A16 axis in the Jura Canton. The tunnel crosses marls and clays formations that may show manifest symptoms of delayed behaviour. The average depth is about 250 m with a maximum overburden of about 400 m in the central part, when the tunnel passes the Aalenian marls.

Particular attention was paid to the geological conditions during tunnel design and excavation. Several excavation methods were used. In the marls, a circular cross section of about 110 m^2 (i.e. equivalent radius, R = 6 m) was excavated with hammers. In order to characterise the potential delayed behaviour of the marls, creep tests and convergence measurements were performed in a reconnaissance gallery.

According to the data collected in the TDB, the temporary support was made of shotcrete reinforced with fibres, rock bolts and reticulated arches, while the definitive lining is a 60 cm thick cast in place high strength reinforced concrete. This choice has been done for guaranteeing the ductility necessary to support potential bending moments and the high pressures that could develop with time. Moreover, due to the aggressive nature of the ground water, rich in sulphates, a total waterproofing system was placed at the lining extrados.

7.2.2 Convergence - Confinement analysis

Basic assumptions of the convergence-confinement method and notations have been described in Chapter 6 (more detailed references can be found also in [126]).

Model parameters

The Mont Terri tunnel is characterised as follows:

- Tunnel depth H = 250 m
- Tunnel radius R = 6 m
- Initial field stress σ_0 = 6.35 MPa
- Temporary support made of fibres reinforced shotcrete: thickness s_s = 0.25 m, elastic modulus E_s = 23 GPa, Poisson's ratio ν_s = 0.2, strength f_{c_s} = 25 MPa
- Distance support from excavation face D_0 = 2 m. Note that the origin of the confinement line, δ_{in}, has been estimated by using the similarity principle introduced by [35]
- Definitive lining made of high strength reinforced concrete: thickness s = 0.6 m, elastic modulus E_c = 35 GPa, Poisson's coefficient ν_c = 0.2, strength f_c = 60 MPa (ductile strain $\varepsilon_u = 0.4\%$)

Viscoplastic behaviour

Excavated rock mass creep can be modelled by a total strain increase:

$$\varepsilon = \varepsilon^{el} + \varepsilon^{vp} \tag{7.12}$$

The elastic strain ε^{el} can be evaluated from the mechanical parameters reported in Table 7.4. Under the assumptions of the convergence-confinement method, it is possible to evaluate the viscoplastic strain ε^{vp} evolution with time by using the Lemaitre's law (i.e. primary creep) for a cylindrical cavity (see Chapter 2 for more details), as reported by [17]:

$$\varepsilon^{vp} = a_{cyl} \cdot (q)^{\beta} \cdot t^{\alpha} \tag{7.13}$$

where t is the time, $q = \sigma_{\theta} - \sigma_r$ the deviatoric stress with axisymmetric loading and plane strain conditions,

$$a_{cyl} = (0.75)^{\frac{\beta+1}{\alpha}} \cdot \left(\frac{A}{\alpha}\right)^{\alpha} \tag{7.14}$$

Two sets of viscoplastic parameters α, β and A have been identified (Table 7.5):

1. the first proposed by [75] results from curve fitting of creep tests,
2. the second proposed by [18] results from curve fitting of measured convergences.

Tab. 7.4. Mechanical parameters for the Aalenian marls (after [18]).

Parameter	Value
γ_{rm}	2.54 $\mathrm{kN/m^3}$
E	8'000 MPa [a]
ν	0.4
φ	20°
c	1 MPa

[a] Note that the present value of Young's Modulus is related to very small elastic deformation as considered in his interpretation.

Tab. 7.5. Viscoplastic parameters for the Mont Terri Aalenian marls as proposed by [75] fitted on creep tests and [18] fitted on in situ convergences.

Parameter	[75]	[18]
A	2E-43	3E-61
α	0.083	0.07
β	1.1	1.1

Knowing the radial stress (i.e. pressure p_i) and tunnel wall displacement (δ_R) it is possible to draw the evolution of the Ground Convergence Curve (GCC) with time t. Deviatoric stress and consequent viscoplastic strain evolution with time have been calculated using the approximate solution proposed by [95] as suggested by [17] (see also [126]). Radial and tangential stresses distributions around the excavation have been computed by integrating Equation 7.15, which describes the equilibrium of a cylindrical cavity under axisymmetrical load and plane strain conditions, starting from $r \gg R$, where both radial and tangential stresses are supposed to be equal to the initial stress σ_0:

$$\frac{\partial\sigma_r}{\partial r} = \frac{q}{r} \tag{7.15}$$

The displacements at the tunnel contour can be evaluated by integrating the total strain (both elastic and viscoplastic) in the space around the excavation.

Results

The top graphs in Figure 7.4 and Figure 7.5 show in a convergence-confinement diagram the tunnel conditions after construction, at 11 years (i.e. the age of the tunnel at present time), and after 30, 50, 70 and 90 years of service life, for the two sets of viscoplastic parameters reported in Table 7.5. In particular, Figure 7.4 shows the evolution of the ground convergence curve using the viscoplastic parameters proposed by [75], and Figure 7.5

using the ones proposed by [18]. Together with the convergence curve, also the tunnel equilibrium, represented by the intersection of the GCC with the confinement line, changes with time due to rock mass ageing. The equilibrium condition can be analysed in terms of Safety Factor (FoS), by comparing the maximum pressure that the supporting structure can bear p_{max} (it is here assumed that the definitive lining properties don't change with time), to the actual pressure exerted by the rock mass and evaluated at the equilibrium point, p_{eq}. The evolution of the Factor of Safety with time is reported on bottom of Figure 7.4 and Figure 7.5.

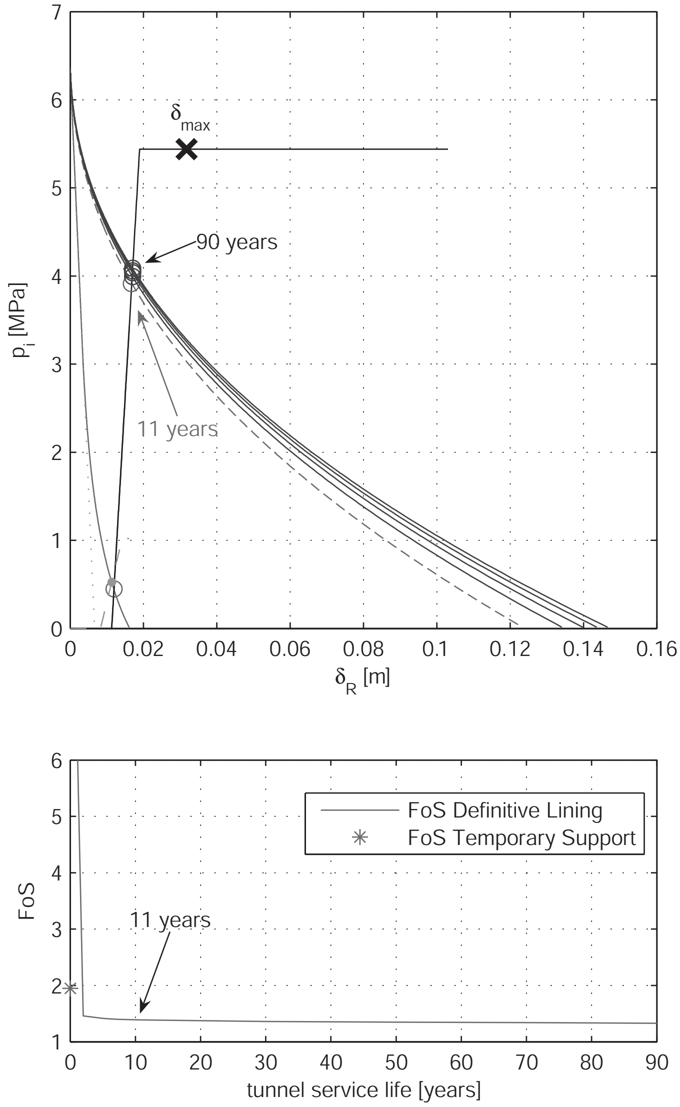


Fig. 7.4. Mont Terri tunnel (Jura Canton, Switzerland). Viscoplastic parameters according to [75]. Tunnel radius $R = 6$ m, definitive lining thickness $s = 0.6$ m. δ_{max} represents the ductility of the lining (see also Chapter 6). On top: Convergence-confinement analysis of tunnel equilibrium evolution with time. The GCC is represented after tunnel construction, at present time (i.e. after 11 years, dashed line), and after 30, 50, 70 and 90 years of tunnel service life. On bottom: Evolution of the Safety Factor FoS with time.

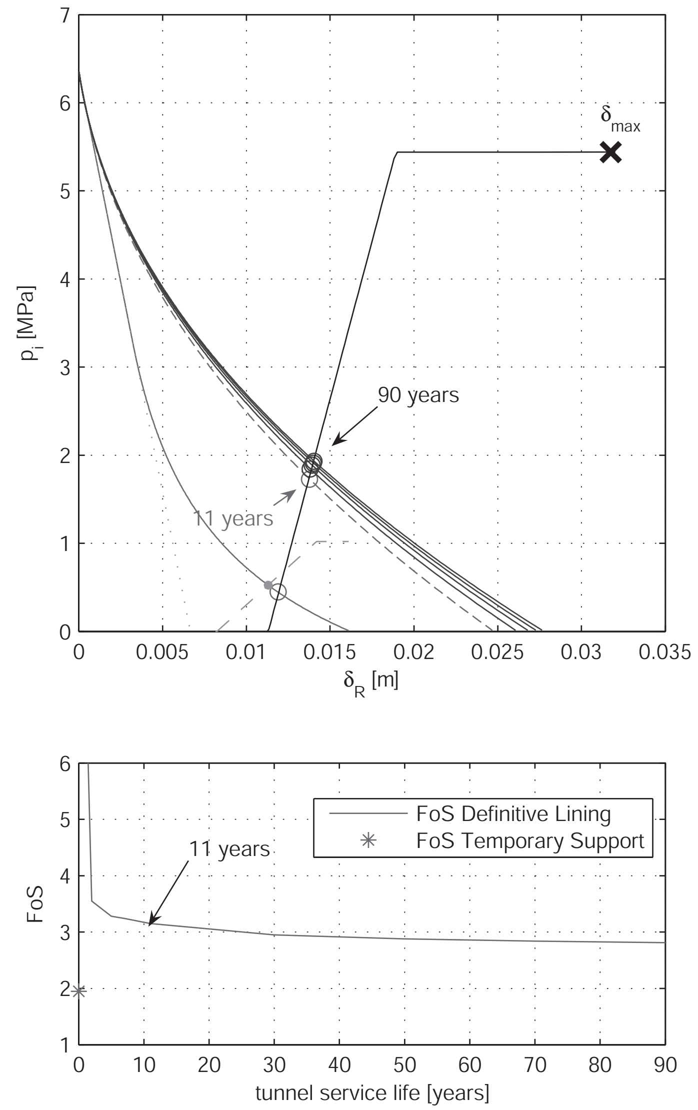


Fig. 7.5. Mont Terri tunnel (Jura Canton, Switzerland). Viscoplastic parameters according to [18]. On top: Convergence-confinement analysis of tunnel equilibrium evolution with time. Tunnel radius $R = 6$ m, definitive lining thickness $s = 0.6$ m. δ_{max} represents the ductility of the lining (see also Chapter 6). The GCC is represented after tunnel construction, at present time (i.e. after 11 years, dashed line), and after 30, 50, 70 and 90 years of tunnel service life. On bottom: Evolution of the Safety Factor FoS with time.

In both cases, under the assumptions of the convergence-confinement method and without considering any degradation to the lining, the equilibrium is satisfied until the end of the tunnel service life. Even in the more unfavourable creep conditions described by the viscoplastic parameters proposed by [75], the lining design can be considered appropriate for assuring tunnel stability at short and long terms.

7.2.3 Conclusions

By comparing the results for the two sets of parameters, it is evident how the evolution of the tunnel equilibrium strongly depends on the chosen parameters for describing the delayed behaviour of the rock mass. A good characterisation of the excavated rock mass is necessary for assessing and interpreting the long term conditions. In this example, remarkable differences have been noted between the characterisation of the rock mass through laboratory tests or in situ measurements. Though in both cases the design choices should satisfy the equilibrium conditions on the long term, it is considered that in situ measurements better represent the rock mass behaviour.

Another important remark should be done concerning the Safety Factor. Its high initial value strongly depends on the basic assumption that assimilates the lining behaviour to a circular concrete ring, under axisymmetric loading conditions, which is rarely met in real cases. However, it is important to note that the Safety Factor has a significant decrease with time due to the delayed behaviour of the rock mass. It is obvious that this decrease due to rock mass ageing should be taken into account already at the design step in order to avoid major problems, all the more as these analyses have been performed without taking into account the lining degradation. Now, in this case, due to the high sulphate content measured in the rock formation during tunnel construction [78], a constant behaviour of the tunnel lining could only be assumed if the waterproofing system works properly during all the service life.

7.3 Long Term Weathering of Concrete Lining - Flonzaley Tunnel

Though in the former example it has been assumed that the tunnel lining doesn't change its behaviour with time, in reality the concrete structure may be affected by long term degradation. In particular, the tunnel lining behaviour changes due to external attacks and chemical weathering [22]; [129]; [33]; [96]. In order to operate effective maintenance, a detailed diagnostic of the identified disorders is required. Verification and laboratory tests may help in estimating the long term effects of tunnel lining pathologies, as it is shown in the following example.

7.3.1 Flonzaley tunnel

At the end of the year 2003, the Flonzaley tunnel (TDB, 2007, Tunnel ID: 277 & 278; Vaud Canton, Switzerland) on the A9 axis, between Lausanne and Chexbres has been object of a verification. Laboratory tests have been performed on specimens bored in the tunnel concrete lining.

The information collected in the Tunnel Data Base together with all information coming from tunnel inspectors (i.e. Perss Ingénieurs conseils SA Fribourg, Switzerland), allow doing the following considerations:

- The tunnel, in operation since the 1973, is quite old and shows symptoms of degradation,
- The concrete lining, made of cast in place concrete (f_{c0} = 45 MPa, at tunnel construction, water to cement ratio W/C = 0.5), has a thickness s of about 0.26-0.30 m,
- A partial waterproofing system has been placed at the lining extrados,
- The tunnel is characterised by a very high daily traffic volume (i.e. more than 50'000 vehicles/day - Mean Daily Traffic Volume).

Moreover, after the tunnel verification performed in 2003, two major concrete lining pathologies have been identified:

1. Concrete weathering caused by water incomes, probably due to a defective waterproofing system,
2. Chlorides penetration.

Lining strength reduction

When the waterproofing system is absent or doesn't work properly (i.e. as supposed in this case since water incomes have been identified as a cause of the concrete weathering), the definitive lining is exposed to external attacks at the tunnel extrados. On the long term, aggressive ground water leaching may affect the mechanical properties of the tunnel lining by increasing the concrete porosity (e.g. sulphates corrosion, calcium leaching...). As it has been described in Chapter 2, [25]; [96] considered that the mechanical properties decrease due to concrete weathering is proportional to the degraded area (i.e. thickness) A_d, for example:

$$\frac{\delta f_c}{f_{c0}} = \frac{f_{c0} - f_c}{f_{c0}} = k_r \cdot \delta A_d \tag{7.16}$$

where $\delta A_d = \frac{A_d}{A_0}$, with $A_d = X_d$, degraded thickness, and $A_0 = s$, and $k_r = 0.76$.

Thanks to laboratory tests performed during tunnel verification, the compressive strength of the concrete has been evaluated after 30 years of operation and compared to its original value given by construction details (i.e. 45 MPa). Using uniaxial compressive tests performed on several samples bored in the tunnel lining in 2003, it results a mean value f_c of about 39 MPa. Thus, as already described in Chapter 2, considering a degraded thickness, X_d [m], proportional to the square root of time, t [days]:

$$X_d = a \cdot \sqrt{t} \tag{7.17}$$

it results $a \cong 5.2\text{E-4} \ \frac{\text{m}}{\text{days}^{0.5}}$ for a medium quality concrete (i.e. water to cement ratio, W/C, equal to 0.5).

Figure 7.6 shows the strength decrease with time according to Equation 7.16.

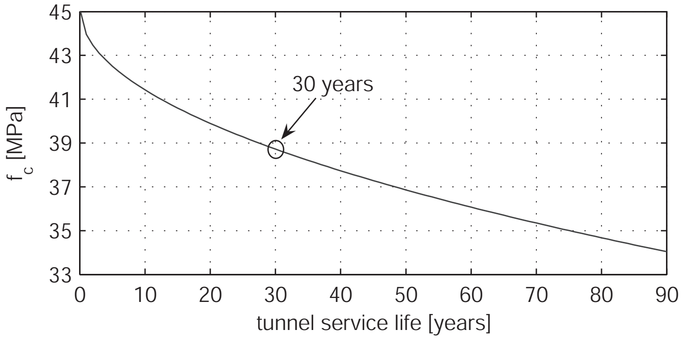


Fig. 7.6. Decrease of compressive strength f_c with time due to aggressive ground water leaching through the tunnel lining. The curve describes the weathering process that affects the concrete lining of the Flonzaley tunnel (Vaud Canton, Switzerland) caused by water incomes due to a defective waterproofing membrane.

De-icing salts aggression - Chlorides corrosion of reinforced concrete structures

As proposed in Chapter 2, by adapting the prescription for bridges of [134], (SN EN 206-1, 2000)] to a typical highway tunnel section, it is possible to identify two different exposure zones for the concrete lining (Figure 2.7, Chapter 2):

- Splash exposure zone that characterises tunnel side walls up to 1.5 m height,
- Mist exposure zone that characterises the upper part of the tunnel walls (i.e. spring line) and the tunnel crown (or the ventilation slab, if case).

In case of reinforced concrete lining the side walls may also be affected by corrosion of reinforcements. This process mainly depends on the amount of free Chlorides that reaches the steel bar. This value can be estimated by means of Chlorides profiles obtained by solving the Fick's equation of diffusion (Equation 2.13, Chapter 2) for different exposure classes at the reinforcement depth. Thus, knowing the reinforcement depth at the tunnel portals (i.e. 50 mm below the lining intrados surface, according to measurements performed during tunnel verification), the free amount of Chlorides has been estimated for a medium quality concrete (i.e. water to cement ratio, W/C, equal to 0.5). In this case, common values used for bridges have been chosen for diffusion coefficient and exposed surface Chloride content, respectively:

- $D_c = 2.14 \cdot 10^{-12}$,
- $C^0_{Cl^-}$ = 3.5 $\frac{kg}{m^3}$ for the splash exposure zone and $C^0_{Cl^-}$ = 2.1 $\frac{kg}{m^3}$ for the mist exposure zone.

Figure 7.7 shows the simulation results and the measured values for the two tunnel portals.

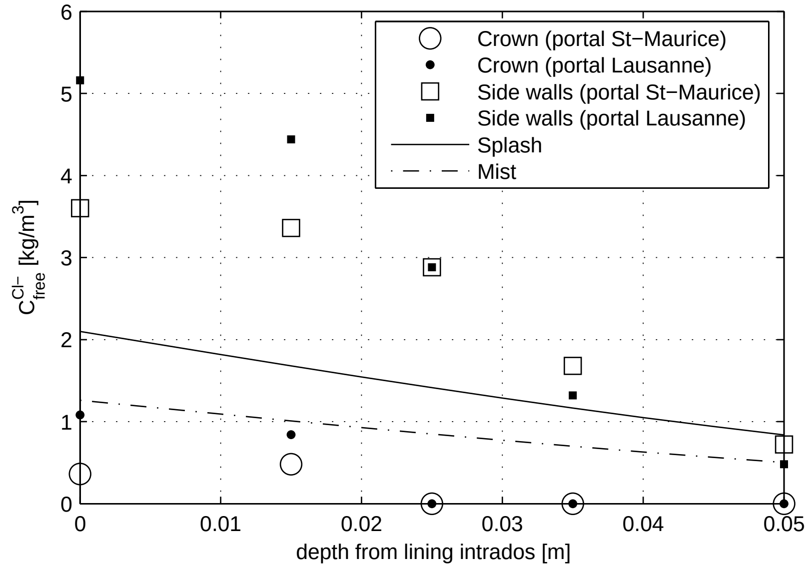


Fig. 7.7. Comparison of estimated and measured Chlorides profiles for the two tunnel portals.

As it is possible to observe, in the splash exposure zone (i.e. tunnel side walls), the results obtained at 50 mm depth represent correctly what has been measured by laboratory tests, while the values in the mist exposure zone (i.e. tunnel crown) seem to be overestimated. This can be explained by the fact that the ventilation system contributes to cleaning the tunnel atmosphere, reducing the de-icing salts concentration.

After evaluating the amount of free Chlorides at the reinforcement depth, as mentioned by [22], it is possible to establish if the corrosion will start with a simple deterministic approach. In particular, for 1 m^3 of reinforced concrete, a conservative limit amount of 0.4% of Chlorides expressed as a fraction (%) of the total cement weight is considered. This value corresponds to a free Chlorides amount of 1.2 $\frac{kg}{m^3}$, for a cement quantity of $300\,\frac{kg}{m^3}$. Thus, as it is possible to observe in Figure 7.7, the risk of corrosion of reinforcements is rather high for the side walls at both tunnel portals.

7.3.2 Conclusions

A correct estimation of the effects of lining pathologies is important for predicting the long term behaviour of the structure and operating effective maintenance. This example shows how initial and operation conditions influence the long term behaviour of the concrete lining. It shows as well how it is possible to use verification results for estimating the evolution of observed disorders together with the reduction of mechanical properties of the tunnel concrete lining.

In the Flonzaley tunnel, it has been possible by means of inspection results to characterise the decrease of mechanical properties for a tunnel lining affected by weathering due to aggressive ground water leaching. Moreover, by means of Chlorides profiles some considerations have been done about the potential corrosion of the reinforced concrete at the tunnel portals.

7.4 Data sources

BGA J. Norbert & B. Schindler (1988): “Tunnel du Mont Russelin - Rapport Géologique, après exécution des forages de reconnaissance” - Service des Ponts et Chaussées section route nationale, Canton du Jura

CMR - Consortium d’Etude Mont Russelin (1990): “N16 - Tunnel du Mont Russelin. Mesures géotechniques” Service des Ponts et Chaussées section route nationale, Canton du Jura

CTB - Consortium Tunnel de Bure (2000): “A16 - Tunnel du Mont Russelin. Evaluation des équipements de mesures géotechniques existantes.” Rapport Août 2000 - Service des Ponts et Chaussées section route nationale, Canton du Jura

Perss Ingénieurs conseils SA - Fribourg, Switzerland (2005): personal communication

Service des Routes Nationales - Canton de Vaud (2005): personal communication

Tunnel Data Base, TDB (2007), Tunnel ID 139 Tunnel Mont Russellin (JU)

Tunnel Data Base, TDB (2007), Tunnel ID 141 Tunnel Mont Terri (JU)

Tunnel Data Base, TDB (2007), Tunnel ID 277&278 Tunnel Flonzaley I, II (VD)

8. Conclusions & Outlook

Due to the mountainous topography, the Swiss National Roads are characterised by a remarkable number of tunnels. Most of these structures has been constructed about 30 years ago and begin requiring interventions in terms of maintenance and conservation.

Compared to railway structures, the degradation of road tunnels, is characterised by new problems, mainly due to a different age of the structures, building material types and construction techniques, geometry and operation conditions.

This work, supported by the Swiss Federal Roads Authority, focuses on the long term behaviour of road tunnels and has been developed into three main stages (with corresponding tools):

- Data collection (Tunnel Data Base),
- Data analysis (G.I.S. and Correspondence Analyses),
- Modelling (degradation models and convergence-confinement method).

TUNNEL DATA BASE. Tunnel durability usually depends on complex interactions between rock mass, ground water and concrete lining, traffic, operational and environmental conditions inside the tunnel, as well as conservation procedures during service life. Information about the main disorders that affect road tunnels, together with detailed data about tunnel construction, operational environment and maintenance procedures are necessary for identifying typical pathologies.

Based on what is already done in other countries (e.g. CETu, France) a data base has been created. After a detailed literature review, together with advices from tunnel inspectors (e.g. people from CETu, France; M. Stempfel from Perss Ingénieurs conseils SA Fribourg, Switzerland; M. Jeanneret from OFROU) a specific technical form for collecting data has been created and sent to each cantonal responsible for National Roads. The majority of data have been collected directly by LMR (F. Sandrone and J.-F. Mathier) consulting cantonal archives for road infrastructures. Moreover, several sources have been compiled to supply information about the tunnels for which it was not possible to get data from cantons. Finally, the data collected with the technical form have been stored in the Swiss Road tunnel data base (TDB) and by means of G.I.S. tools it has been possible to supply to the lack of information especially for what concerns geological and hydrogeological conditions.

At present, the TDB contains detailed information about 168 tunnels of the Swiss National Road Network. Apart from general information, it is possible to identify three main sections of data related to the whole tunnel life (see also Table 4.9):

- Construction,
- Environment and Operation,
- Maintenance (note that this information is available only for 122 tunnels).

The TDB may be considered as an effective tool for providing a general description of the Swiss National Roads tunnels conditions. Moreover, it allows to identify the main features of the National Roads tunnels that are necessary for assessing their future behaviour.

DATA ANALYSIS. After collecting data both about tunnel features and observed disorders, tunnel degradation potential and rate have been analysed in order to identify main causes and influence factors. Typical exploratory data analysis techniques combined with G.I.S. tools were used.

The agreement of G.I.S. analyses results and observed disorders allows considering G.I.S. as a promising tool for the preliminary detection of tunnels potential pathologies. An interesting feature of using G.I.S. tools is the possibility of representing on maps all the information stored in the Tunnel Data Base for an easier identification of problems related to the tunnel location (e.g. geological and hydrogeological conditions). Anyway, though these tools may help for a preliminary detection of tunnel pathologies, the results clearly show the need for improving the detail scale of the collected data for better characterising the structures and their potential disorders.

The contribution of each tunnel feature to tunnel degradation has been described and analysed separately. In particular, it has been shown how tunnel age, construction methods and materials, waterproofing and ventilation systems, together with depth, geological and hydrogeological conditions, determine the initial equilibrium of the tunnel and its degradation potential. While, operation conditions, tunnel environment and chemical aggression by weathering agents, such as ground water and de-icing salts, clearly influence the tunnel degradation rate.

By means of Correspondence Analyses, it has been possible to operate a first selection of influence factors that mainly characterise tunnel operation conditions. Moreover, by estimating the disorders probability, the influence of selected factors (or combination of factors) on the development of certain pathologies has been pointed out and described.

This analysis shows that lining disorders, identified during tunnel inspections, may be caused by rock mass degradation and delayed behaviour, as well as, tunnel environment and operation conditions (e.g. traffic, de-icing salts). For instance it has been shown how:

- High traffic volumes, together with the use of de-icing salts can be particularly aggressive for the concrete lining. Indeed, lining weathering and corrosion due to de-icing salts attack mainly affect the lower part of the tunnel side walls.
- The chemical composition of ground water, together with the absence of waterproofing system, the poor quality of building materials and the aggressive atmosphere inside the tunnel accelerate lining weathering rate. The main symptoms of this pathology are efflorescence and concrete corrosion (e.g. sulphates attack, calcium leaching...) that affect mainly tunnel crown.
- The effects of squeezing potential due to bad geological conditions, and developing at a certain depth, may be reduced by an appropriate tunnel design (e.g. supporting structure and tunnel geometry).

Finally, the analysis clearly shows the need of specifying some recommendations in order to facilitate the assessment of tunnel long term behaviour. In particular, it has been pointed out how more refined geological information together with detailed inspection results (reported at the tunnel local scale for example by means of simple sketches or, better, based on laser scanner profiles and camera recording during tunnel principal inspections) are necessary for a better diagnosis of the actual tunnel conditions.

MODELLING. After the identification of the main degradation processes and causes, the tunnel stability evolution with time has been modelled by taking into account the long term behaviour of both rock mass and lining. A specific methodology is described, for evaluating the tunnel conditions during its service life. By a regular update of input parameters, long term effects can be considered and tunnel equilibrium conditions evaluated at a certain time after its construction. Several examples of application are presented in the framework of the convergence-confinement method. In particular, simplified degradation models have been used for representing:

- the delayed behaviour of the excavated rock mass (i.e. ageing),
- weathering of both rock mass and lining due to the aggressive action of ground water, pollution and de-icing salts (i.e. hydrogeological and operational conditions)
- the additional effects of consolidation processes (pore water pressure redistribution) depending on the hydraulic boundary condition (i.e. tunnel waterproofing system).

In order to estimate the evolution of the tunnel stability, the results have been interpreted in terms of Safety Factor (or Factor of Safety,), by comparing the resistance (strength) of the lining structure, represented by maximum pressure that the supporting structure can bear, and the effect of action (stress) in it, which corresponds to the actual pressure exerted by the rock mass.

The results indicate clearly that both rock mass and lining degradations cause a performance decrease of the tunnel. Moreover, they point out the importance of checking tunnel safety and serviceability both at short (i.e. during/after construction) and at long (i.e.during operation) terms.

In particular, it has been shown that the delayed behaviour of rock masses (either with a viscoelastic or viscoplastic behaviour) may strongly affect the long term Safety Factor while lining weathering shows more moderate effects. Thus, in order to avoid major problems, appropriate rheological models for describing the rock mass long term behaviour should be chosen already at the tunnel conception and design stage.

Due to the restrictive assumptions on which the convergence-confinement method is based (circular tunnel, homogeneous ground, hydrostatic initial stresses, etc.), the main purpose of its application is to help in better understanding the influence of the tunnel degradation on the long term equilibrium and not to replace detailed calculations and investigations by the tunnel management responsible. The methodology described in this work should be adapted to more refined FEM-calculations.

The results of this work clearly illustrate the necessity of taking into account both rock mass and lining degradation for better assessing the long term behaviour of tunnels.

Anyway, some limitations identified in the present work may become, in the Author's opinion, starting points for further research. In particular:

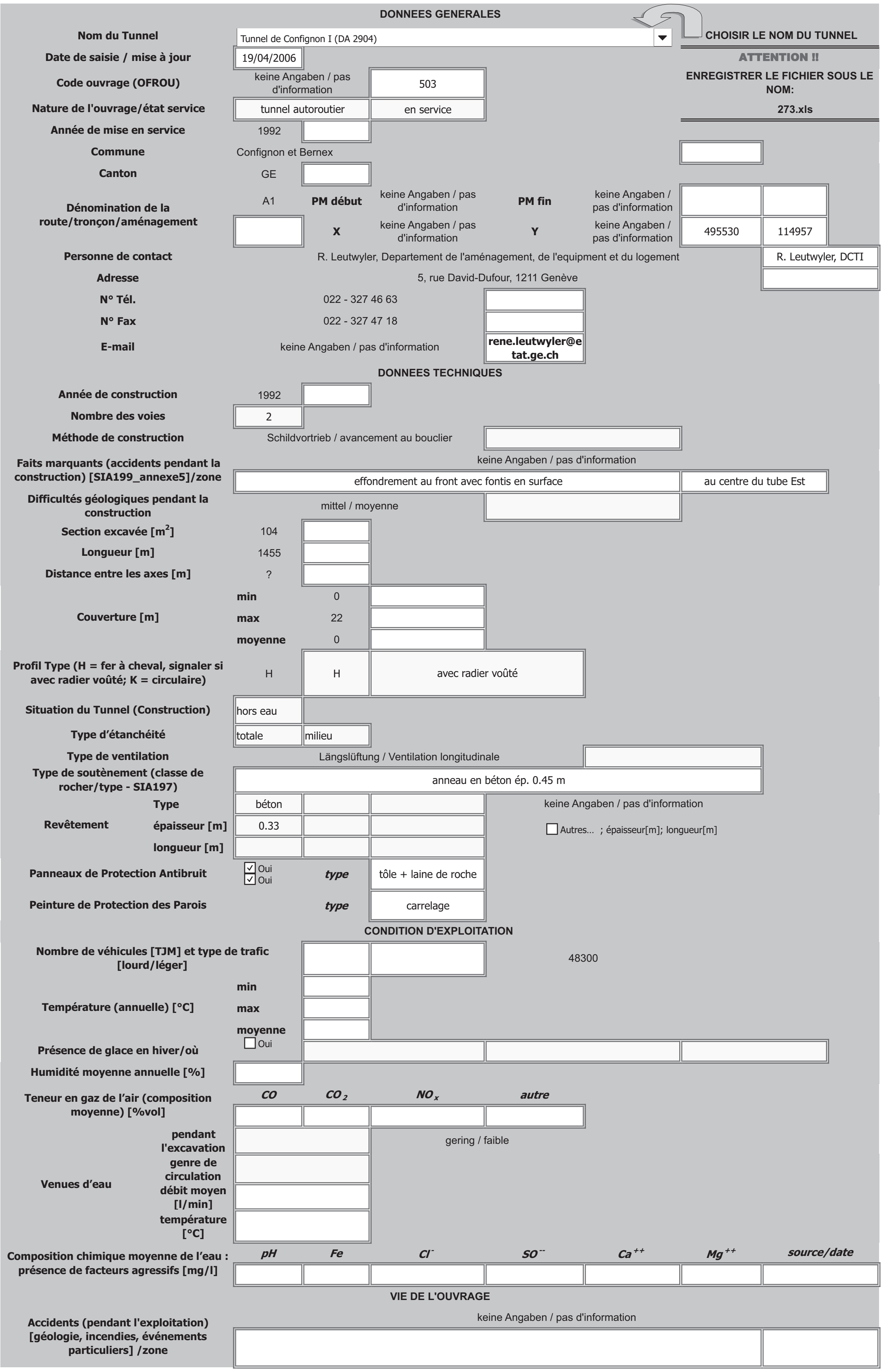
- This work shows how, by means of data analysis, it is possible to identify the causes (i.e. combination of geological, geotechnical, environmental, structural and operational/ maintenance conditions) that lead to disorders development and to a decrease of the tunnel serviceability level. Moreover, it shows clearly that inspections results and disorders observations, together with a detailed characterisation of the tunnel at the local scale are necessary for correctly interpret tunnel degradation phenomena. In this thesis, the lack of data (mainly geological and hydrogeological) has been supplied by G.I.S. information which comes from national offices. For developing an effective tool to help with tunnels maintenance decisions, detailed inspections results, together with geological and hydrogeological characterisation, at the tunnel local scale, are still required. Therefore, the possibility of recovering detailed information would allow to refine the data analysis and improve the investigation of influence factors that contribute to tunnel degradation. This should improve the pathologies identification for a better diagnosis of the tunnel conditions and a more efficient maintenance. Moreover, a detailed geological information should allow to improve the rock mass modelling by means of appropriate constitutive laws and mechanical parameters.
- Effective repair procedures still need more precise evaluation of actual stability conditions and assessment of remaining strength. In order to take into account the long term effects on tunnel stability conditions it is necessary to define the rate (i.e. characteristic time) of degradation processes (i.e. ageing, weathering.) of both the rock mass and the lining. At present, available information about successive inspections of Swiss Road tunnels is not enough detailed for estimating the time evolution of observed disorders. By means of a constant update of the collected data, it should be possible to determine the tunnel degradation rate in a more precise way. Furthermore, for assessing disorders evolution with time, another possible improvement is based on monitoring measurements interpretation. For instance, the interpretation of lining deformations measured with a laser scanner profilometer, should allow an estimation of stresses changes in the structure and help in foreseeing lining failure.
- Even though the presented work brings about new understandings of tunnel degradation, the proposed analysis based on the convergence-confinement method is restricted by its basic assumptions. As a matter of fact, the local character of several pathologies could not be taken into consideration; moreover, the perfect axial-

symmetric situation is rarely met in real cases. Therefore, to evaluate the influence of long term degradation on tunnel equilibrium conditions and verify the estimations presented in this work, it would be necessary to extend the proposed methodology to other calculation methods such as reactions method or finite element modelling. Together with a more detailed characterisation of both the structure (i.e. rock mass and lining) and the disorders, those methods should allow to refine the results presented in this thesis.

Annexes

I Technical Form

I.1 Example of technical form filled by cantonal responsibles (GE).



DONNEES GENERALES

CHOISIR LE NOM DU TUNNEL

ATTENTION !!

ENREGISTRER LE FICHIER SOUS LE NOM:

273.xls

Nom du Tunnel	Tunnel de Confignon I (DA 2904)						
Date de saisie / mise à jour	19/04/2006						
Code ouvrage (OFROU)	keine Angaben / pas d'information	503					
Nature de l'ouvrage/état service	tunnel autoroutier	en service					
Année de mise en service	1992						
Commune	Confignon et Bernex						
Canton	GE						
Dénomination de la route/tronçon/aménagement	A1	**PM début**	keine Angaben / pas d'information	**PM fin**	keine Angaben / pas d'information		
		X	keine Angaben / pas d'information	**Y**	keine Angaben / pas d'information	495530	114957
Personne de contact	R. Leutwyler, Departement de l'aménagement, de l'equipment et du logement	R. Leutwyler, DCTI					
Adresse	5, rue David-Dufour, 1211 Genève						
N° Tél.	022 - 327 46 63						
N° Fax	022 - 327 47 18						
E-mail	keine Angaben / pas d'information	**rene.leutwyler@etat.ge.ch**					

DONNEES TECHNIQUES

Année de construction		1992		
Nombre des voies		2		
Méthode de construction		Schildvortrieb / avancement au bouclier		
Faits marquants (accidents pendant la construction) [SIA199_annexe5]/zone		keine Angaben / pas d'information	effondrement au front avec fontis en surface	au centre du tube Est
Difficultés géologiques pendant la construction		mittel / moyenne		
Section excavée [m²]		104		
Longueur [m]		1455		
Distance entre les axes [m]		?		
Couverture [m]	**min**	0		
	max	22		
	moyenne	0		
Profil Type (H = fer à cheval, signaler si avec radier voûté; K = circulaire)		H	H	avec radier voûté
Situation du Tunnel (Construction)		hors eau		
Type d'étanchéité		totale	milieu	
Type de ventilation		Längslüftung / Ventilation longitudinale		
Type de soutènement (classe de rocher/type - SIA197)		anneau en béton ép. 0.45 m		
Revêtement	**Type**	béton		keine Angaben / pas d'information
	épaisseur [m]	0.33		☐ Autres... ; épaisseur[m]; longueur[m]
	longueur [m]			
Panneaux de Protection Antibruit	☑ Oui ☑ Oui	***type***	tôle + laine de roche	
Peinture de Protection des Parois		***type***	carrelage	

CONDITION D'EXPLOITATION

Nombre de véhicules [TJM] et type de trafic [lourd/léger]				48300
Température (annuelle) [°C]	**min**			
	max			
	moyenne			
Présence de glace en hiver/où	☐ Oui			
Humidité moyenne annuelle [%]				

Teneur en gaz de l'air (composition moyenne) [%vol]	***CO***	***CO_2***	***NO_x***	***autre***

Venues d'eau	**pendant l'excavation**		gering / faible
	genre de circulation		
	débit moyen [l/min]		
	température [°C]		

Composition chimique moyenne de l'eau : présence de facteurs agressifs [mg/l]	***pH***	***Fe***	***Cl^-***	***SO^{--}***	***Ca^{++}***	***Mg^{++}***	***source/date***

VIE DE L'OUVRAGE

Accidents (pendant l'exploitation) [géologie, incendies, événements particuliers] /zone	keine Angaben / pas d'information	

ENTRETIEN

Inspections	**Année de la dernière visite principale**	2004	***fréquence***	1fois/5ans
	Année de la dernière visite intermédiaire	2005	***fréquence***	1fois/an

Surveillance Auscultations (instrumentations)

type	***Zone ™***
mesures de convergence	tout le tunnel
suivi venues d'eau	tout le tunnel
contrôle ponctuel du système de drainage	tout le tunnel

Maintenance

intervention	***fréquence***	***Zone ™***
lavage du revêtement	1fois/6mois	tout tunnel
brossage parois	1fois/6mois	tout tunnel
pompage du système de drainage des eaux	1fois/6mois	tout tunnel
curage du réseau de canalisation des eaux de surface	1fois/6mois	tout tunnel
vidage du système de dessablage et séparateur d'huile	1fois/6mois	tout tunnel

Entretien

Revêtement du Tunnel/Galerie:

pathologie	***cause***	***intervention***	***date***	***Zone ™***
fissures		aucune		

Revêtement de la Chaussée:

pathologie	***cause***	***intervention***	***date***	***Zone ™***

Bordures:

pathologie	***cause***	***intervention***	***date***	***Zone ™***
corrosion des armatures et taches de rouille (bordures)	saumure	aucune		

Système de Drainage:

pathologie	***cause***	***intervention***	***date***	***Zone ™***

Rénovation

type d'intervention	***faits marquants***	***date***	***Zone ™***

DONNEES GEOLOGIQUES

Description de principaux types de roches rencontrés (unités lithologiques/géologiques - SIA199)

Sedimentgestein / roche sédimentaire

molasse , cailloutis et moraine

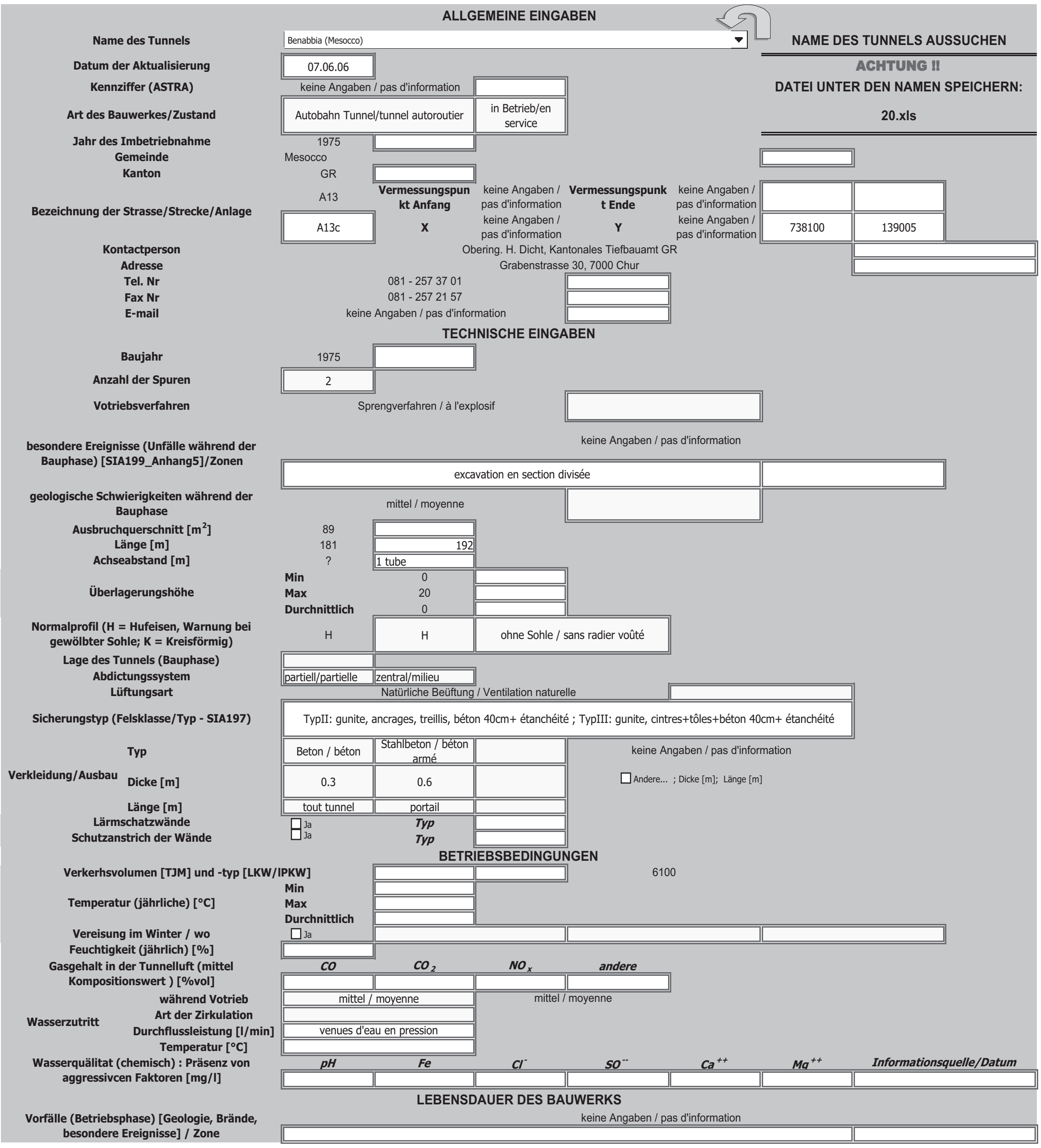
Données géol. plus détaillées

Profil en long géologique du tunnel et du massif ☑ Oui ***format*** papier ***où*** DCTI

HYDROGEOLOGIE

Présence de la nappe/niveau ☑ Oui variable

I.2 Example of technical form filled by LMR - EPFL (F. Sandrone & J.-F. Mathier).



ALLGEMEINE EINGABEN

NAME DES TUNNELS AUSSUCHEN

ACHTUNG !!

DATEI UNTER DEN NAMEN SPEICHERN:

20.xls

Name des Tunnels	Benabbia (Mesocco) ▼						
Datum der Aktualisierung	07.06.06						
Kennziffer (ASTRA)	keine Angaben / pas d'information						
Art des Bauwerkes/Zustand	Autobahn Tunnel/tunnel autoroutier	in Betrieb/en service					
Jahr des Imbetriebnahme	1975						
Gemeinde	Mesocco						
Kanton	GR						
Bezeichnung der Strasse/Strecke/Anlage	A13	**Vermessungspunkt Anfang**	keine Angaben / pas d'information	**Vermessungspunkt Ende**	keine Angaben / pas d'information		
	A13c	**X**	keine Angaben / pas d'information	**Y**	keine Angaben / pas d'information	738100	139005
Kontactperson	Obering. H. Dicht, Kantonales Tiefbauamt GR						
Adresse	Grabenstrasse 30, 7000 Chur						
Tel. Nr	081 - 257 37 01						
Fax Nr	081 - 257 21 57						
E-mail	keine Angaben / pas d'information						

TECHNISCHE EINGABEN

Baujahr		1975		
Anzahl der Spuren		2		
Votriebsverfahren		Sprengverfahren / à l'explosif		
besondere Ereignisse (Unfälle während der Bauphase) [SIA199_Anhang5]/Zonen		keine Angaben / pas d'information excavation en section divisée		
geologische Schwierigkeiten während der Bauphase		mittel / moyenne		
Ausbruchquerschnitt [m^2]		89		
Länge [m]		181	192	
Achseabstand [m]		?	1 tube	
Überlagerungshöhe	**Min**		0	
	Max		20	
	Durchnittlich		0	
Normalprofil (H = Hufeisen, Warnung bei gewölbter Sohle; K = Kreisförmig)		H	H	ohne Sohle / sans radier voûté
Lage des Tunnels (Bauphase)				
Abdictungssystem		partiell/partielle	zentral/milieu	
Lüftungsart		Natürliche Beüftung / Ventilation naturelle		
Sicherungstyp (Felsklasse/Typ - SIA197)		TypII: gunite, ancrages, treillis, béton 40cm+ étanchéité ; TypIII: gunite, cintres+tôles+béton 40cm+ étanchéité		

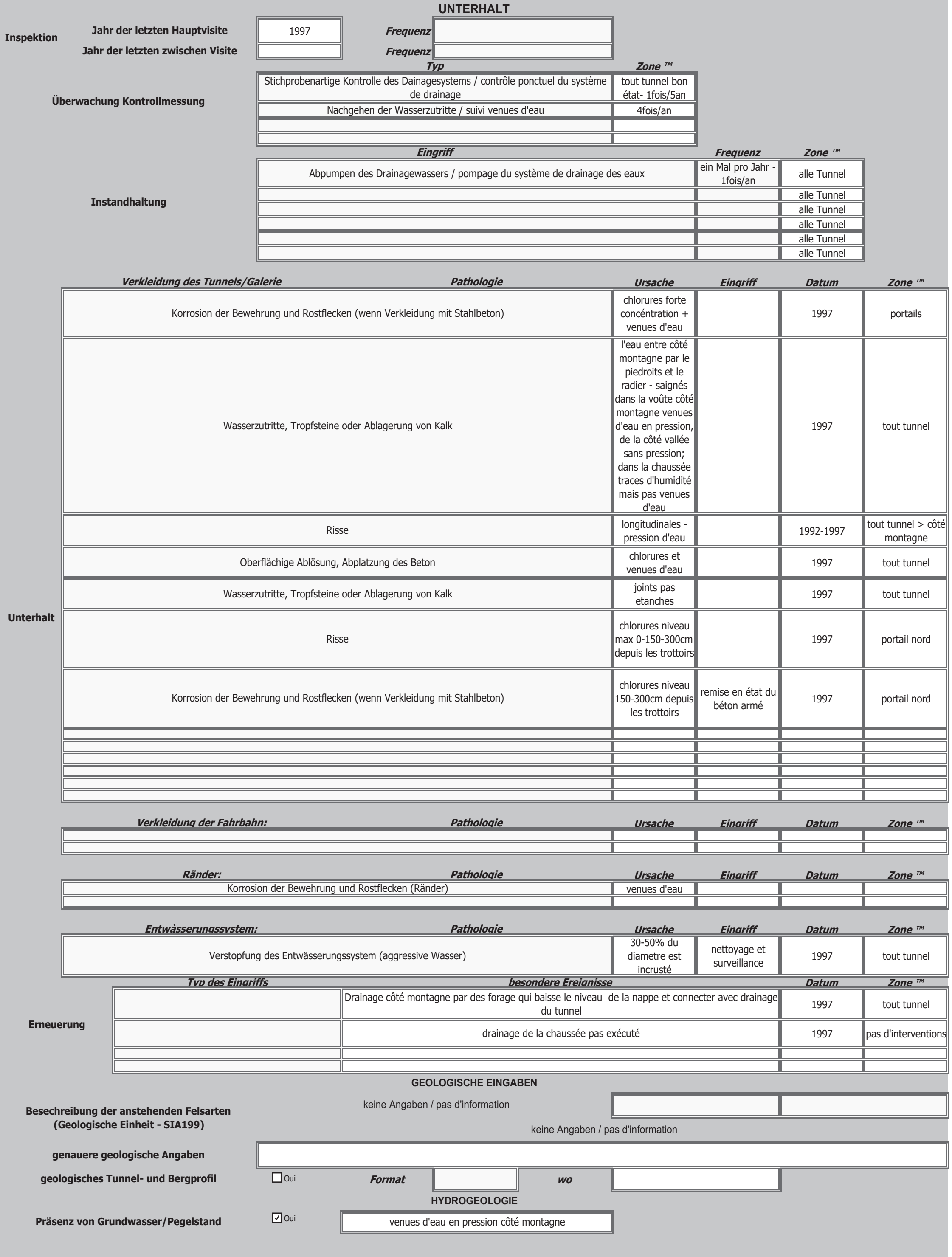
Verkleidung/Ausbau	**Typ**	Beton / béton	Stahlbeton / béton armé		keine Angaben / pas d'information
	Dicke [m]	0.3	0.6		☐ Andere... ; Dicke [m]; Länge [m]
	Länge [m]	tout tunnel	portail		
Lärmschatzwände		☐ Ja	***Typ***		
Schutzanstrich der Wände		☐ Ja	***Typ***		

BETRIEBSBEDINGUNGEN

Verkerhsvolumen [TJM] und -typ [LKW/IPKW]					6100		
Temperatur (jährliche) [°C]	**Min**						
	Max						
	Durchnittlich						
Vereisung im Winter / wo	☐ Ja						
Feuchtigkeit (jährlich) [%]							
Gasgehalt in der Tunnelluft (mittel Kompositionswert) [%vol]	***CO***	***CO_2***	***NO_x***	***andere***			
Wasserzutritt – **während Votrieb**	mittel / moyenne		mittel / moyenne				
Wasserzutritt – **Art der Zirkulation**							
Wasserzutritt – **Durchflussleistung [l/min]**	venues d'eau en pression						
Wasserzutritt – **Temperatur [°C]**							
Wasserqülitat (chemisch) : Präsenz von aggressivcen Faktoren [mg/l]	***pH***	***Fe***	***Cl^-***	***SO^{--}***	***Ca^{++}***	***Mg^{++}***	***Informationsquelle/Datum***

LEBENSDAUER DES BAUWERKS

Vorfälle (Betriebsphase) [Geologie, Brände, besondere Ereignisse] / Zone	keine Angaben / pas d'information



UNTERHALT

Inspektion

Jahr der letzten Hauptvisite	1997	***Frequenz***	
Jahr der letzten zwischen Visite		***Frequenz***	

Überwachung Kontrollmessung

Typ	*Zone ™*
Stichprobenartige Kontrolle des Dainagesystems / contrôle ponctuel du système de drainage	tout tunnel bon état- 1fois/5an
Nachgehen der Wasserzutritte / suivi venues d'eau	4fois/an

Instandhaltung

Eingriff	*Frequenz*	*Zone ™*
Abpumpen des Drainagewassers / pompage du système de drainage des eaux	ein Mal pro Jahr - 1fois/an	alle Tunnel
		alle Tunnel
		alle Tunnel
		alle Tunnel
		alle Tunnel
		alle Tunnel

Unterhalt

Verkleidung des Tunnels/Galerie — *Pathologie*	*Ursache*	*Eingriff*	*Datum*	*Zone ™*
Korrosion der Bewehrung und Rostflecken (wenn Verkleidung mit Stahlbeton)	chlorures forte concéntration + venues d'eau		1997	portails
Wasserzutritte, Tropfsteine oder Ablagerung von Kalk	l'eau entre côté montagne par le piedroits et le radier - saignés dans la voûte côté montagne venues d'eau en pression, de la côté vallée sans pression; dans la chaussée traces d'humidité mais pas venues d'eau		1997	tout tunnel
Risse	longitudinales - pression d'eau		1992-1997	tout tunnel > côté montagne
Oberflächige Ablösung, Abplatzung des Beton	chlorures et venues d'eau		1997	tout tunnel
Wasserzutritte, Tropfsteine oder Ablagerung von Kalk	joints pas etanches		1997	tout tunnel
Risse	chlorures niveau max 0-150-300cm depuis les trottoirs		1997	portail nord
Korrosion der Bewehrung und Rostflecken (wenn Verkleidung mit Stahlbeton)	chlorures niveau 150-300cm depuis les trottoirs	remise en état du béton armé	1997	portail nord

Verkleidung der Fahrbahn: — *Pathologie*	*Ursache*	*Eingriff*	*Datum*	*Zone ™*

Ränder: — *Pathologie*	*Ursache*	*Eingriff*	*Datum*	*Zone ™*
Korrosion der Bewehrung und Rostflecken (Ränder)	venues d'eau			

Entwàsserungssystem: — *Pathologie*	*Ursache*	*Eingriff*	*Datum*	*Zone ™*
Verstopfung des Entwässerungssystem (aggressive Wasser)	30-50% du diametre est incrusté	nettoyage et surveillance	1997	tout tunnel

Erneuerung

Typ des Eingriffs	*besondere Ereignisse*	*Datum*	*Zone ™*
	Drainage côté montagne par des forage qui baisse le niveau de la nappe et connecter avec drainage du tunnel	1997	tout tunnel
	drainage de la chaussée pas exécuté	1997	pas d'interventions

GEOLOGISCHE EINGABEN

Besechreibung der anstehenden Felsarten (Geologische Einheit - SIA199): keine Angaben / pas d'information

keine Angaben / pas d'information

genauere geologische Angaben:

geologisches Tunnel- und Bergprofil ☐ Oui ***Format*** ***wo***

HYDROGEOLOGIE

Präsenz von Grundwasser/Pegelstand ☑ Oui venues d'eau en pression côté montagne

II TDB: Factors / Modalities List

Tab. II.1. Swiss Tunnel Data Base Structure (after [128])

Section	**Data**	**Données**	**Daten**
General Information Informationes Générales Allgemeine Informationen	Tunnel name	Nom du tunnel	Name des Tunnels
	Town, Canton	Commune, Canton	Gemeinde, Kanton
	Road	Dénomination de la route	Bezeichnung der Strasse
	Local Operator	Personne de contact	Kontaktperson
	Commissioning (operation) year	Année de mise en service	Jahr der Inbetriebnahme
	Coordinates X,Y (centre point)	Coordonnées X, Y (centre)	Koordinaten X,Y (Mittelpunkt)
	Lane No.	Nombre de voies	Anzahl der Spuren
Construction Information Details de construction Baueingaben	Construction Year	Année de construction	Baujahr
	Geometrical Data (depth, length, section size, interaxis)	Données Géometriques (profondeur, longueur, taille de la section, entraxes)	geometrische Daten (Überlagerungshöhe, Länge, Ausbruchquerschnitt, Achsabstand)
	Excavation method	Méthode d'excavation	Vortriebsverfahren
	First support (type and length along the tunnel)	Soutènement (type et longueur)	Sicherungstyp / Stützwerk (Typ, Länge)
	Definitive lining (type, thickness and length)	Revêtement (type, épaisseur, longueur)	Verkleidung (Typ, Länge, Dicke)
	Waterproofing and drainage	Type d'étanchéité et système de drainage	Abdichtungssystem, Entwässerungssystem
	Accidents during construction	Faits marquants (accidents) pendant la construction	besondere Ereignisse (Unfälle während der Bauphase)
	Geological profile and description	Profile géologique et description	geologisches Tunnel- und Bergprofil
	Geological difficulties during excavation	Difficultés géologiques pendant la construction	geologische Schwierigkeiten während der Bauphase

Section	Data	Données	Daten
Environment and Operation Information Conditions d'exploitation Betriebsbedingungen	Accidents during operation	Accidents pendant l'exploitation	Vorfälle (Betriebsphase)
	Traffic	Trafic	Verkehr
	Temperature	Température	Temperatur
	Humidity	Humidité	Feuchtigkeit
	Chemical composition of tunnel atmosphere	Teneur en gaz de l'air	Gasgehalt in der Tunnelluft
	Chemical composition of groundwater	Composition chimique de l'eau	Wasserquälitat (chemisch)
	Groundwater level and circulation type	Niveau de la nappe et type de circulation	Präsenz von Grundwasser und Pegelstand
	Technical equipment (ventilation)	Equipement technique (ventilation)	Lüftungsart
Maintenance Information Entretien Unterhalt	Inspection (date and frequency)	Visite principale (date et fréquence)	Hauptvisite (Datum und Frequenz)
	Monitoring	Auscultations	Überwachung und Kontrollmessung
	Routine maintenance	Maintenance (ordinaire)	Instandhaltung
	Disorders (date of observation, possible cause, area and eventual repair)	Désordres (date de l'observation, cause possible, zone et intervention)	Pathologie (Datum, Ursache, Zone, Eingriff)
	Renewal /Refurbishment (intervention date and type, area, cause)	Rénovation (type d'intervention, date, zone, cause)	Erneuerung (Typ des Eingriffs, Datum, Zone, besondere Ereignisse)

In order to maximise the consistency between tunnels for the analysis, for each factor (i.e. variable) several modalities/attributes have been identified and coded as shown in the following table:

Tab. II.2. List of variable and modalities used for the data collection in the TDB.

Factor	Modality/Attribute	Modalité	Modalität
Construction/ Commissioning Year	<1970 (A)	A	A
	1970-1990 (B)	B	B
	>1990 (C)	C	C

Factor	**Modality/Attribute**	**Modalité**	**Modalität**
Construction Method	Full face shield (Sh)	avancement au bouclier	Schildvortrieb
	TBM (TBM)	tunnelier	TBM
	Drill & Blast (DB)	à l'explosif	Sprengverfahren
	Road-header (Rh)	MAP (attaque ponctuelle)	TSM
	Road-header under shield protection (RhS)	MAP sous bouclier	TSM mit Schild
Tunnel Geometry - section size (m^2)	<70	<70	<70
	70-90	70-90	70-90
	90-110	90-110	90-110
	>110	>110	>110
Tunnel Geometry - section shape	Circular (K)	ciculaire	Kreisprofil
	Horseshoe (H)	fer à cheval sans radier voûté	Hufeisen ohne Sohle
	Horseshoe with invert (HR)	fer à cheval avec radier voûté	Hufeisen mit Sohle
Waterproofing system	No waterproofing (K)	aucune	keine
	Partial in the middle (Pm)	partielle, milieu	partiell, zentral
	Partial extrados (Pe)	partielle, extrados	partiell, äussere Wölbung
	Total middle (Tm)	totale, milieu	gesamt, zentral
	Total extrados (Te)	totale, extrados	gesamt, äussere Wölbung
	Tunnel intrados (Ti)	intrados	Gewölbelaibung
Depth (m)	<80	<80	<80
	80-500	80-500	80-500
	>500	>500	>500
Geology	Carbonate rocks (S_ce) Evaporites (S_ce+) Marls & Clayey media (S_cm) Schists & Phyllites (M_sp) Crystalline formation (M_g) Loose Ground (L)	calcaires evaporites marnes, argiles schistes et phillites cristallin terrain meuble	Kalksteine Evaporit Mergel, Ton Schiefer kristallines Gestein Lockergestein
Geological difficulties during construction	Slight (+)	faible	gering
	Medium (++)	moyenne	mittel
	Medium to difficult (++/+++)	moyenne - forte	mittel - stark
	Difficult (+++)	forte	stark
Water incomes during construction	Slight (+)	+	+
	Important (++)	++	++
	Very important (+++)	+++	+++

Factor	**Modality/Attribute**	**Modalité**	**Modalität**
Water circulation around excavation	Karstified (K)	karst	Karst
	Porous (P)	pores	Porecirculazion / porös
	Discontinuous (D)	discontinuités	Diskontinuität
First Support type	Shotcrete with fibres	Béton projété fibré	Stahlfaserspritzbeton
	Shotcrete without fibres	Béton projété	Spritzbeton
	Rock bolts	Boulons	Anker / Bolzen
	Steel sets	Cintres en acier	Stahlbogen
	Injection	Injections	Injektionen
	Jetting	Jetting	Jetting
	Concrete	Béton	Beton
Definitive Lining type	Concrete	Béton	Beton
	Reinforced Concrete	Béton armé	Stahlbeton
	Precast Concrete	Béton préfabriqué	Vorfabrizierter Beton
	Prestressed Concrete	Béton précontraint	Vorgerspannter Beton
Definitive Lining thickness (m)	<0.25	<0.25	<0.25
	0.25 - 0.40	0.25 - 0.40	0.25 - 0.40
	0.40 - 1.0	0.40 - 1.0	0.40 - 1.0
	>1.0	>1.0	>1.0
Tunnel Ventilation system	Natural (N)	ventilation naturelle	Natürliche Belüftung
	Longitudinal (L)	ventilation longitudinale	Längslüftung
	Transverse (Q)	ventilation transversale	Quer-Lüftung
	Semi-transverse (HQ)	ventilation semi-transversale	Halbquer-Lüftung
Accidents during construction	Instability - collapse - settlement	Instabilité, collapse, tassement	Instabilität, Setztung
	Unexpected water inflow	Irruption d’eau	Bergwasserausbruch
	Other...	Autre..	Andere..
Accidents during operation	Accident (car collision, fire...)	Accident (collisions des voitures, incendies)	Vorfälle (Brände, besondere Ereignisse)
	Instability - collapse - settlement	Instabilité, collapse, tassement	Instabilität, Setztung
	Other...	Autre..	Andere..

Factor	**Modality/Attribute**	**Modalité**	**Modalität**
Tunnel part (area/ zone affected by disorders, accidents and/or object of repair)	Crown	Voûte	Gewölbe
	Drainage System	Système de drainage	Entwässerungssystem
	Face	Front	Ortsbrust
	General	générale	Allgemein
	Gutter	Trottoirs - Chaussée	Bankett - Fahrbahn
	Invert	Radier	Sohle
	Portals	Portails	Portale
	Pavement	Chaussée	Fahrbahn
	Ventilation slab	Plafonds (système de ventilation)	Lüftungsdecken
	Side walls	Piédroits	Tunnelwand
	Side wall foundation	Base de piédroit	Wandfundament
Disorder type	Water leakage, Moisture ***[1]***	Humidité, Infiltration d'eau	Wasserzutritte, Tropfsteine
	Efflorescence (sulphates) ***[2]***	Efflorescence (corrosion par sulfates)	Ausblühungen des Betons
	Staining, Calcium leaching effects, Calcareous concretion, Honeycomb ***[3]***	Concrétion calcaires, altération et lessivage du béton, Nids de gravier	Ablagerung von Kalk, Zerfall des Betons Kiesneste
	Concrete spalling, Delaminated concrete (reinforcements corrosion) ***[4]***	Eclatement du béton d'enrobage (corrosion du béton armé)	Oberflächige Ablösung, Abplatzung des Beton (Chloridkorrosion)
	Plain concrete corrosion, Concrete scaling (de-icing salts attack) ***[5]***	Corrosion du béton, écaillage causé par les sels de déverglaçage	Abplatzungen oder Ablösung der Platten durch übermäßige Spannungen
	Corrosion of steel bars (reinforced concrete) ***[6]***	Corrosion des renforcements du béton armé	Korrosion der Bewehrung und Rostflecken (wenn Verkleidung mit Stahlbeton)
	Voids behind the lining ***[7]***	Zones sonnant creux (vides derrière le revêtement)	Zonen die hohl klingen
	Cracks, Fissures ***[8]***	Fissures	Risse
	Local deformation (crown) ***[9]***	Déformation locale de la voûte	Lokale oder flächendeckende Verformungen im Gewölbe
	Local deformation (walls) ***[10]***	Déformation locale des piédroits	Lokale oder flächendeckende Verformungen im Widerlager

Factor	**Modality/Attribute**	**Modalité**	**Modalität**
	Fines transport in the drainage system ***[11]***	Transport des fines dans le système de drainage	getriebenen oder abgestzten Materialien
	Ice formation ***[12]***	Formation de glace	Zersetzung der Wände durch den Frost, den Fall von Eisblöcke
	Concrete lining crumbling - local failure, blocks fall ***[13]***	Effritement du revêtement, ruptures locales et chute de blocs	Getrennte oder unstabile Elemente
	Track scaling ***[14]***	Faïençage de la chaussée	Abblätterung der Fahrbahn-verkleidung
	Invert heave up ***[15]***	Soulèvement du radier	Lokale oder flächendeckende Verformungen in der Sohle
	Impact damages ***[16]***	Endommagement par collisions et chocs	Beschädigung durch Aufprallen bei Unfälle
	Drainage system obstruction by calcareous concretion ***[17]***	Obstruction du système de drainage par des concrétions calcaires	Verstopfung des Entwässerungssystem (aggressive Wasser)
Principal and/or Secondary Inspection frequency	once / 6 months	1 fois /6 mois	einmal / alle sechs Monate
	once / 1 year	1 fois / an	einmal pro Jahr
	once / 2 year	1 fois / 2 ans	einmal alle zwei Jahre
	once / 3 year	1 fois / 3 ans	einmal alle drei Jahre
	once / 4 year	1 fois / 4 ans	einmal alle vier Jahre
	once / 5 year	1 fois / 5 ans	einmal alle fünf Jahre

Factor	**Modality/Attribute**	**Modalité**	**Modalität**
Monitoring (intervention type)	Water incoming measurements	suivi venues d'eau	Nachgehen der Wasserzutritte
	Convergences measurements	mesures de convergence	Konvergenzmesswerte
	Ice formation control	contrôle de formation de glaçons	Kontrolle der Eisbildung
	Drainage system local inspection	contrôle ponctuel du système de drainage	Stichprobenartige Kontrolle des Entwässerungssystem
	Levelling measurements	Mesures de nivellation	Nivellierung
	Cracks monitoring	Auscultation des fissures	Kontrolle auf Risse
	Stresses measurements / Pressure cells	Mesure des contraintes, cellules de pression	Spannungsmessung, Druckmessung
	Strain measurements / Extensometer	Mesures des deformations, extensomètres	Deformationsmessung, Extensometer
	Profilometer	Auscultation du profil	Profilometer
	Concrete investigation and Chlorides corrosion	Investigations du béton, mesure de la corrosion par chlorures	mechanische Versuche an Beton, Tiefe der Chlorid-Korrosion
	Other...	Autre..	Andere..
Maintenance (intervention type)	Tunnel washing	lavage du revêtement	Reinigung der Verkleidung
	Clean any debris from walls and gutter	brossage parois	Abbürsten der Wände
	Drain flushing	pompage du système de drainage des eaux	Abpumpen des Drainagewassers
	Clean through any surface water drainage	curage du réseau de canalisation des eaux de surface	Reinigung des Kanalnetzes zur Abführung
	Other...	Autre..	Andere..

Factor	**Modality/Attribute**	**Modalité**	**Modalität**
Repair (intervention type)	Water channelling to avoid ice formation and water inflow	traitement de formation de glaçons (captages)	Behandlung der Eisbildung (Erfassung)
	Track repair	rabotage chaussée, réfection revêtement de la chaussée usée	Abschleifen der Fahrbahn, Instandsetzung des Fahrbahnbelags
	Lining repair	réparation des piédroits	Instandsetzung der Tunnelwand
	Drainage system repair	réparation du système de drainage	Zustand der Kanalisation Entwässerungssystem
	Internal waterproofing system (repair)	système d'étanchéité à l'intrados	Gewölbelaibung
	Gutter and side walls foundations repair	réfection des fondations des piédroits	Instandsetzung der Wandfundament
	Investigation and verification	Investigations et verification	Überprüfung
	No intervention	aucune	Keine
	Other...	autre..	Andere..
Renewal (intervention type)	Lining Reinforcement / renewal	Réinforcement du revêtement / renovation	Erneuerung und Verstärkung der Verkleidung
	Ventilation system renewal	Réfection du système de ventilation	Erneuerung der Tunnellüftung
	Electro-mechanical system renewal	Renovation de l'equipement électro-mécanique	Erneuerung der Tunnelbetriebstechnik
	Other...	Autre..	Andere..

III Glossary in alphabetical order

Concept	Meaning
Acceptance Test / First Inspection, “Réception de l’ouvrage” [100] (Chapter 3)	First principal inspection after tunnel construction, made for verifying the initial quality level of the structure.
Ageing, “Vieillissement” (Chapter 2)	Time dependent behaviour of the structure. It consists in physical degradation processes that affect both the rock mass and the lining.
Arch Action, “Effet voûte” [143] (Chapter 2)	Capacity of the rock located above the tunnel crown to transfer the major part of the total overburden weight onto the rock located on both sides of the tunnel.
Calcium leaching, “Lixiviation du béton” [96] (Chapter 2)	Concrete degradation caused by weathering action of ground water, with consequent loss in mechanical properties.
Categorical Variable, “Variable catégorique ou qualitative” (Chapter 5)	Also called nominal variable, is a variable that has two or more categories/attributes (or modalities) without intrinsic order. For example, tunnel shape is a categorical variable having three categories (i.e. horseshoe shaped, horseshoe shaped with invert and circular) and there is no intrinsic ordering to the categories.
Conservation, “Conservation” [132]; [147] (Chapter 3)	All the procedures for maintaining the tunnel in serviceable and safe conditions and preserving its value.
Contingency table, “Table de contingences” (Chapter 5)	Table used for analysing relationships between two or more categorical variables.
Control, “Contrôle” (Chapter 3)	It is performed during inspection for checking different parameters and identifying pathologies
Convergence, “Convergence” [3] (Chapter 2)	Variation of the distance between two points located at the tunnel intrados.
Convergence-Confinement Method (Cv-Cf Method), “Méthode convergence-confinement” [3] (Chapter 6)	Analytical method for representing the interaction between the excavated rock mass and the lining. It represents the behaviour of both the rock mass (described by the Cv line or Ground Convergence Curve, GCC) and the supporting structure (described by the Cf line) in terms of loads vs. radial wall displacements.
Correspondence Analysis, “Analyse des correspondances” [60]; [92] (Chapter 5)	Multivariate statistics exploratory technique which allows to analyse contingency tables and describe relationships between rows and columns data.

Concept	Meaning
Creep, "Fluage" (Chapter 2)	Increasing strain while the stress is held constant.
Daily Traffic Volume (D.T.V.), "Trafic Journalier Moyen (T.J.M.)" (Chapter 5)	Average value of number of vehicles per day passing through a tunnel.
Damage, "Dégât" [100]; [131] (Chapter 2)	Tunnel serviceability and/or safety level decrease due to external and/or internal causes.
Internal Damage, "Dégât interne" [10] (Chapter 2)	Deterioration of the internal structure of the concrete (even without visible external damage) which leads to a change in the concrete properties (e.g. reduction in elastic modulus).
Defect, "Défaut" [100]; [131]; [30] (Chapter 2)	Lack of the required quality level.
Degradation, "Dégradation" (Chapter 1)	Tunnel serviceability and/or safety level reduction due to long term changes of mechanical properties and/or behaviour caused by damaging factors.
Delayed Behaviour, "Comportement différé" (Chapter 2)	Evolutive behaviour typical of certain rock formations.
Deterioration, "Détérioration" (Chapter 2)	Decay. Tunnel evolutive deteriorating and weakening processes caused by damaging factors.
Diagnosis, "Diagnostic" (Chapter 3)	Analysis and evaluation of the tunnel global conditions. Identification of main pathologies and causes, estimation of their evolution rate and suggestion of appropriate conservation procedures.
Disorder, "Désordre" [30] (Chapter 2)	Lack or disturbance in the expected quality level of a tunnel caused by internal or external damaging factors. Pathology symptom.
Durability, "Durabilité" (Chapter 2)	Aptitude of a tunnel to resist to external loads and actions during a certain period without significant changes in its ordinary characteristics.
Extrados, "Extrados" (Chapter 3)	Outer part or external surface, of the tunnel supporting structure (i.e. interface with excavated rock mass or tunnel waterproofing system).
Future Life Span, "Durée d'utilisation restante" [131] (Chapter 3)	Future life expectancy, without changing operational requirements.

Concept	**Meaning**
Honeycomb, “Nid de gravier” (Chapter 2)	Area or portion of hardened concrete which is deficient in mortar and consisting primarily of coarse aggregate and open voids. Usually it is caused by insufficient vibration during casting process, it can also be the result of calcium leaching from the concrete structure with increase of porosity of the matrix.
Infrastructure, “Infrastructure” [101] (Chapter 1)	In the case of a tunnel, it is composed of excavated rock-mass, lining and electro-mechanical equipment
Inspection, “Inspection” [147]; [133] (Chapter 3)	Periodical activities for controlling tunnel global conditions, detecting disorders and following its time evolution. It can be general or detailed (principal) characterised by different frequency, control type. Moreover, different techniques can be used in depending on the control type.
Intrados, “Intrados” (Chapter 2)	The inner part or internal surface, of the tunnel supporting structure (i.e. visible and inspectable part of the tunnel).
Maintenance, “Maintenance” [132] **/ “Entretien Courant”** [100] (Chapter 3)	All the simple and routine procedures (i.e. with a precise scheduling and a defined duration) to increase durability of the tunnel, without changing the tunnel performances. It can also be a preventive action.
Monitoring, “Contrôle par mesures” [132] **/ “Auscultation”** [100] (Chapter 3)	Regular measurements to keep under control the evolution of disorders identified during principal inspections.
Observation, “Observation” (Chapter 3)	Sort of operational supervision by means of regular (i.e. daily, weekly, monthly, yearly) visits. It allows recognizing local and surface degradations.
Operation, “Exploitation” (Chapter 1)	All the features that characterise the ordinary use of a tunnel, specified at the design stage (e.g. for example, in the case of a road tunnel, traffic conditions).
Pathology, “Pathologie” [30] (Chapter 2)	Problem that causes tunnel disorders, relation between the disorder and its cause.
Performance, “Performance” (Chapter 2)	Aptitude of a structure to resist to the external actions, for a tunnel it is a function of both initial (e.g. geological conditions, geometry, material quality, construction method) and operation conditions.
Phenomenon, “Evènement” (Chapter 2)	Anything that can be observed. Degradation effect, pathology symptom, disorder.

Concept	**Meaning**
Refurbishment, “Rénovation” [132] **/ “Renouvellement”** [100] (Chapter 3)	Important structural reinforcement after a failure or at the end of the service life.
Rehabilitation, “Remise en état” [132] **/ “Gros Entretien”** [100] (Chapter 3)	Conservation procedures to improve the tunnel affected by considerable disorders and re-establish a good service level.
Renewal, “Rénovation” [132] **/ “Renouvellement”** [100] / “Modification” [147] (Chapter 3)	Tunnel rebuilding or transformation to new operational requirements at the end of its service life. Any kind of activity that transform the original structure at the end of its service life.
Repair, “Entretien Courant” [132] **/ “Gros Entretien”** [100] (Chapter 3)	see Rehabilitation.
Road network, “Réseaux routières” [147] (Chapter 1)	All road infrastructures that can be used by both motorised and not motorised traffic.
Swiss National Roads, “Routes Nationales Suisses” (Chapter 1)	All roads that belong to the Swiss Confederation.
Safety Factor (FoS**), “Factor de securité”** (Chapter 6)	Resistance of the tunnel supporting structure vs. external load (equilibrium conditions).
Scaling, “Ecaillage” [10]; [74], (Chapter 2)	Removing of thin layer from the concrete lining surface with consequent decrease of the initial thickness. This process can be the result of chemical actions of freeze-thaw de-icing agents attack.
Service Life, “Durée de service” (Chapter 2)	Period of the whole tunnel life when all the design functions are accomplished and the structure has an adequate performance level.
Serviceability, “Aptitude au service” [133] (Chapter 1)	Aptitude of the tunnel to operate in normal conditions as defined at the design stage, without major defects (Serviceability Limit State).
Shrinkage, “Retrait” (Chapter 2)	Time dependent strain result of concrete ageing without any external action and/or stress. Contraction and extension process which causes cracks formation.
Squeezing rock, “Roche poussante” (Chapter 2)	“Large time-dependent convergence during tunnel excavation. It takes place when a particular combination of induced stresses and material properties pushes some zones around the tunnel beyond the limiting shear stress at which creep starts. Deformation may terminate during construction or continue over a long period of time”. [13]

Concept	Meaning
Structural Element, "Ouvrage" (Chapter 3)	In the case of a tunnel, it is represented by both excavated rock-mass and supporting structure.
Structural Safety, "Sécurité structurale" [133] (Chapter 3)	Aptitude of the tunnel to guarantee the global stability avoiding tunnel failure (Ultimate Limit State).
Survey, "Surveillance" [100]; [146] (Chapter 3)	All activities that allow to follow the tunnel evolution with time (i.e. observation, inspection and monitoring).
Swelling rock, "Roche gonflante" (Chapter 2)	Delayed behaviour of rock which tends to increase its volume with time, simultaneously with the moisture content.
Total Traffic Volume (T.T.V) / Traffic Charge, "Volume total du trafic" (Chapter 5)	Total amount of vehicles passed through the tunnel since its commissioning year (i.e. during the whole tunnel service life).
Urgent Repair, "Mesure urgente" [100] **/ "Mesures urgentes de sécurité"** [133] (Chapter 3)	Urgent procedure to re-establish an adequate safety level in tunnel operation.
Verification, "Vérification" [100] (Chapter 3)	Detailed investigation of zones and parts of the tunnel where some defects/problems have been identified, it is necessary for a good diagnosis of the problem and an effective repair.
Weathering, "Altération" (Chapter 2)	Chemical degradation process caused by weathering agents (e.g. air and ground water). It causes reduction of tunnel mechanical properties (i.e. both rock mass and concrete).
Work of Art, "Ouvrage d'art" [103] (Chapter 1)	A structure whose construction requires a specific technical knowledge.

IV List of Symbols

Symbol	**Meaning**
Chapter 1	No symbols used.
Chapter 2	
$\varepsilon_{t_0}^{el}$	Instantaneous elastic strain (Concrete ageing [28])
t_0	Initial instant of time
t	Time
ε_t	Strain (Concrete ageing [28])
ε^c	Creep strain (Concrete ageing [28])
$\bar{\varphi}(t, t_0)$	Creep coefficient (Concrete ageing [28])
σ_{t0}	Stress at time t_0 (Concrete ageing [28])
E	Young's Modulus
σ	Stress
E_{t0}	Young's Modulus at time t_0
φ_0	Function of the concrete quality (Concrete ageing [28])
β_E	Coefficient for evaluating Young's Modulus with time (Concrete ageing [28])
$\beta_{t_{0(0)}}$	Function of the loading time (Concrete ageing [28])
β_{RH}	Function of both relative humidity and temperature (Concrete ageing [28])
β_σ	Function dependent (only if $(0.4 \cdot f_c) \leq \sigma \leq (0.6 \cdot f_c)$) (Concrete ageing [28])
f_c	Compressive strength of concrete aged of 28 days
β_t	Coefficient for evaluating the evolution of creep with time (Concrete ageing [28])
J	Flux (Fick's diffusion law)
D	Diffusivity (Fick's diffusion law)
$C(x, t)$	Concentration gradient (Fick's diffusion law)
X_d	Thickness of the degraded zone (Weathering)
a	Material constant (Concrete weathering [25]; [96])
δA_d	Increase of the degraded area (Concrete weathering [25]; [96])
A_d	Degraded area (Concrete weathering [25]; [96])
A_0	Initial section (Concrete weathering [25]; [96])

Symbol	**Meaning**
f_{c0}	Initial compressive strength (Concrete weathering [25]; [96])
k_m	Material constant (Concrete weathering [25]; [96])
k_r	Material constant (Concrete weathering [25]; [96])
D	Mechanical damage variable [90]
V	Chemical damage variable, function of Calcium ions concentration in the pore solution
$C_{Cl^-}^{free}$	Free Chlorides content
$C_{Cl^-}^{0}$	Exposed surface Chloride content
x	Depth from concrete surface
D_c	Diffusion coefficient
erf	Error function (from solving Fick's second law of diffusion)
R_{oxi}	Material constant for rock mass weathering
R	Tunnel radius
R_{pl}	Plastic radius
Δp_{crown}	Load increase on the tunnel crown [111]
γ_{rm}	Rock mass unit weight
σ_c	Rock compressive strength
K_p	Passive earth pressure coefficient
K_{psi}	Dilatancy coefficient
ψ	Dilatancy angle ($0 \le \psi \le \varphi$)
φ	Friction angle (Mohr-Coulomb criterion)
c	Cohesion (Mohr-Coulomb criterion)
ε	Total strain
ε_{el}	Elastic strain
ε_{vp}	Viscoplastic strain
$g(\sigma_{ij})$	Function of stress
$f(t)$	Function of time
$\dot{\varepsilon}_{vp}$	Viscoplastic strain rate (Lemaitre's law)
q	Deviatoric stress in the rock mass (Lemaitre's law)

Symbol	**Meaning**
F_0	Reference stress (= 1 MPa) (Lemaitre’s law)
σ_s	Limit stress beyond which the delayed behaviour starts (Lemaitre’s law)
n	Stress power - material constant ($n > 1$, Lemaitre’s law)
m	a constant ($(1-n) < m < 0$, Lemaitre’s law)
A	Viscosity parameter, described by Arrhenius’s law (Lemaitre’s law)
A_0	Material constant (Arrhenius’s law)
R	Universal gas constant (Arrhenius’s law)
T	Absolute temperature (Arrhenius’s law)
ΔG_0	Activation energy of thermic reaction (Arrhenius’s law)
α	Time power (Lemaitre’s law for a cylindrical cavity [17])
β	Stress power (Lemaitre’s law for a cylindrical cavity [17]),
a	material constant (Lemaitre’s law for a cylindrical cavity [17])
σ_0	Initial stress state, ratio of in-situ stresses
H	Overburden, tunnel depth
σ_{cm}	Rock mass compressive strength
p_i	Internal support pressure
$\Delta\varepsilon_v$	Volumetric strain increase (Swelling, [Gysel, (1987) see [23])
ε_v	Volumetric strain (Swelling, [Gysel, (1987) see [23])
ε_r	Radial strain
ε_θ	Tangential strain
p	Average stress, $p = \dfrac{\sigma_r + \sigma_\theta + \sigma_z}{3}$
σ_r	Radial stress
σ_θ	Tangential stress
σ_z	Longitudinal stress
ν	Poisson’s ratio
p_g	Maximum swelling pressure (Swelling, [23])
k_g	Free swelling (Swelling, [23])

Symbol	Meaning
σ_g	Swelling pressure (Swelling, [23])
C_g	Swelling index introduced by Huder-Amberg in 1970 (Swelling, [23])
σ_{v0}	Initial vertical stress in oedometrical test (Swelling, [23])
σ	Total stress (Terzaghi's criterion)
u	Interstitial pressure (Terzaghi's criterion)
σ'	Effective stress (Terzaghi's criterion)
Chapter 3	No symbols used.
Chapter 4	No symbols used.
Chapter 5	
DTV	Daily Traffic Volume
γ_{rm}	Rock mass unit weight
σ_0	Field stress
σ_{cm}	Rock mass compressive strength
σ_{ci}	Intact rock compressive strength
STD	Standard Deviation
ε	Strain
$\frac{\sigma_{cm}}{\sigma_0}$	Competency factor (Squeezing potential [69])
p_i	Internal support pressure
GSI	Geological Strength Index (GSI_{max}, GSI_{min} , respectively maximum and minimum values)
n_{rows}	Number of rows (Correspondence Analysis)
$m_{columns}$	Number of columns (Correspondence Analysis)
N	Axial load (thrust)
M	Bending moment
Chapter 6	
FoS	Factor of Safety
p_{max}	Maximum that the supporting structure can bear
p_{eq}	Equilibrium pressure
p_i	Pressure acting on the tunnel lining (Convergence line)
p_s	Pressure acting on the tunnel lining (Confinement line)

Symbol	**Meaning**
H	Tunnel depth
γ_{rm}	Rock mass unit weight
R	Tunnel Radius
σ_0	Initial stress field
σ_r	Radial stress
σ_θ	Tangential stress
pp_0	Initial pore water pressure (New Design Method, [58])
δ_r	Radial displacements
δ_R	Radial displacement at the tunnel contour
δ_{in}	Initial displacement at lining installation
q	Deviatoric stress in cylindrical coordinates (axisymmetrical load & plane strain conditions)
t	Time
E	Young's Modulus of the rock mass
ν	Poisson's ratio of the rock mass
E_c	Young's Modulus of the concrete lining
E_s	Young's Modulus of the shotcrete support
ν_c	Poisson's ratio of the concrete lining
ν_s	Poisson's ratio of the shotcrete support
s	Lining thickness
s_s	Support thickness
f_c	Uniaxial compressive strength of concrete
f_{c_s}	Uniaxial compressive strength of shotcrete
k_s	Stiffness of the supporting structure
φ	Friction angle (Mohr-Coulomb criterion)
c	Cohesion (Mohr-Coulomb criterion)
ψ	Dilatancy angle ($0 \le \psi \le \varphi$)
ε_r	Radial strain
ε_θ	Tangential strain

Symbol	**Meaning**
r	Radial distance from the tunnel wall
p_{cr}	Critical pressure
R_{pl}	Plastic radius
s	Lining thickness
p_s	Pressure at supporting structure extrados (Confinement line)
D_0	Distance of the section where the support is constructed from the tunnel face
D	Normalised distance of the support from the tunnel face $D = \frac{D_0}{\chi}$ [35]
δ_{pl}	Final convergence of an unlined tunnel excavated in an elastic-plastic medium
δ_{el}	Final convergence of an unlined tunnel excavated in an elastic medium
χ	Similarity ratio [35]
$\varepsilon_0, \varepsilon_1, \varepsilon_2$	Three limit strains [125]
C, C_0	Peak and residual cohesions [125]
η	Viscosity
n	Stress power (Viscoplasticity)
p_g	Maximum swelling pressure (Swelling, [23])
ε_v^p	Plastic volumetric strain around the tunnel (Weathering, [26])
ε_u	Ductile strain of concrete [28]
$\delta_R(t)$	Radial wall displacement with time (Viscoelasticity)
$\delta_{t_0}^{el}$	Elastic displacement at time t_0 (Viscoelasticity)
G_0	Short term shear modulus, $G_0 = \frac{E_0}{2 \cdot (1 + \nu_0)}$ (Viscoelasticity)
E_0	Short term Young's Modulus of the rock mass (Viscoelasticity)
ν_0	Short term Poisson's ratio of the rock mass (Viscoelasticity)
t_0	Initial instant of time
T	Relaxation time (Viscoelasticity)
G_1	Long term shear modulus (Viscoelasticity)
G_∞	Viscoelastic shear modulus $G_\infty = \frac{G_0 \cdot G_1}{G_1 + G_0}$ (Kelvin-Voigt model)

Symbol	**Meaning**
$\dot{\varepsilon}$	Total strain rate
$\dot{\varepsilon}^{e}$	Elastic strain rate
$\dot{\varepsilon}^{vp}$	Viscoplastic strain rate
ε^{vp}	Viscoplastic strain
α	Time power [17]
β	Stress power [17]
A	Viscosity parameter [17]
a_{cyl}	Viscosity parameter in a cylindrical cavity $a_{cyl} = (0.75)^{\frac{\beta+1}{\alpha}} \cdot \left(\frac{A}{\alpha}\right)^{\alpha}$ [17]
ε_{vp_θ}	Tangential viscoplastic strain
ε_{vp_r}	Radial viscoplastic strain
Δp_{crown}	Load increase on the tunnel crown [111]
K_w	Water bulk modulus (New Design Method)
k_{s0}	Initial value of support stiffness (Support degradation)
p_{max0}	Initial value of maximum support pressure (Support degradation)
X_d	Thickness of the degraded zone (Weathering)
a	Material constant [25]; [96]
V_{ST}	Short term strength parameter (Rock mass weathering)
V_{LT}	Long term strength parameter, 70% of the V_{ST} (Rock mass weathering)
T	Constant that defines the rate of the rock mass weathering
Chapter 7	
δ_{crown}, δ_1	Measured displacement at tunnel crown
$\delta_{invert}, \delta_2$	Measured displacement at tunnel invert
δ_{left}, δ_3	Measured displacement at tunnel left side wall
δ_{right}, δ_4	Measured displacement at tunnel right side wall
δ_m	Mean displacement
δ_v	Mean vertical displacement
δ_h	Mean horizontal displacement

Symbol	**Meaning**
δ_0	Lateral displacement at tunnel invert
p_{lat}	Rock mass reaction to tunnel shape deformation
E	Young's Modulus of the rock mass
ν	Poisson's ratio of the rock mass
R	Tunnel radius
p_1	External load (asymmetric loading [42] after [93])
p_2	External load (asymmetric loading [42] after [93])
p_v	Mean vertical pressure
p_h	Mean horizontal pressure
M_{sw}	Bending moment at side wall
M_c	Bending moment at tunnel crown
N_{sw}	Thrust at tunnel side wall
N_c	Thrust at tunnel crown
E_c	Young's Modulus of the concrete lining
ν_c	Poisson's ratio of the concrete lining
s	Lining thickness
I	Inertia of the lining section $I = \frac{s^3}{12}$
σ	Stress in the lining (at intrados I and extrados E)
f_c	Uni-axial compressive strength of the concrete
H	Tunnel depth
s_s	Temporary support thickness
E_s	Young's Modulus of the temporary support
ν_s	Poisson's ratio of the temporary support
f_{c_s}	Uni-axial compressive strength of the temporary support
D_0	Distance support from excavation face
δ_{in}	Initial displacement at lining installation
σ_0	Initial stress field
p_i	Pressure acting on the tunnel lining

Symbol	Meaning
δ_R	Radial displacement at the tunnel contour
ε_u	Ductile strain of the concrete lining
φ	Friction angle (Mohr-Coulomb criterion)
c	Cohesion (Mohr-Coulomb criterion)
ψ	Dilatancy angle ($0 \le \psi \le \varphi$)
ε	Total strain
ε^{el}	Elastic strain
ε^{vp}	Viscoplastic strain
t	Time
α	Time power [17]
β	Stress power [17]
A	Viscosity parameter [17]
a_{cyl}	Viscosity parameter for a cylindrical cavity $a_{cyl} = (0.75)^{\frac{\beta+1}{\alpha}} \cdot \left(\frac{A}{\alpha}\right)^{\alpha}$ [17]
q	Deviatoric stress in cylindrical coordinates (axisymmetrical load & plane strain conditions)
r	Radial distance from the tunnel wall
σ_r	Radial stress
FoS	Factor of Safety
p_{max}	Maximum that the supporting structure can bear
p_{eq}	Equilibrium pressure
f_{c0}	Compressive strength of the sound material [25]; [96]
δA_d	Increase of the degraded area [25]; [96]
k_r	Material constant [25]; [96]
A_d	Degraded area [25]; [96]
X_d	Thickness of the degraded zone (Weathering)
a	Material constant [25]; [96]
D_c	Diffusion coefficient
$C^0_{Cl^-}$	Exposed surface Chloride content

Abbreviations

Abbreviation	Meaning
AAR	Alkali-Aggregate Reaction
ASTRA	Bundesamt für Strassen
BDR	Roads Data Base)
CEB-FIP	Euro-International Concrete Committee (CEB - Comité Euro-International du Béton) and International Federation for Prestressing (FIP - Fédération Internationale de la Précontrainte)
CFF	Chemins de fer fédéraux suisses
CETu	Centre d'études des tunnels
Cv-Cf	Convergences-Confinement Method
CMR	Consortium d'Etude Mont Russelin
DATEC	Dipartimento federale dell'ambiente, dei trasporti, dell'energia e delle comunicazioni
DETEC	Département fédéral de l'environnement, des transports, de l'énergie et de la communication
DTM	Digital Terrain Model
DTV	Daily Traffic Volume
EN	European Norm
ENPC	École National des Ponts et Chaussées
EPFL	École Polytechnique Fédéral de Lausanne
ETHZ	Eidgenössische Technische Hochschule Zürich
FEDRO	Federal Roads Office
FGU	Fachgruppe für Untertagbau
GCC	Ground Characteristic Curve
GIS	Geographic information system
GSI	Geological Strength Index
INERIS	Institut national de l'environnement Industriel et des risques
ITA-AITES	International Tunnelling And Underground Space Association
KUBA-DB	Kunstbautendatenbank der Nationalstrassen
KUBA-MS	Kunstbautenmanagementsystem
LMR	Laboratoire de mécanique des roches
MATUF	Recommendation system in maintenance and repairing of tunnels (Portugal)
OFEV	Office fédéral de l'environnement
OFROU	Office fédéral des routes
OFS	Office fédéral de la statistique
RADIS	Data base for tunnel inspection (France)
RH	Relative humidity

RN	Routes Nationales
SIA	Société suisse des ingénieurs et des architectes
SIG	Società Italiana Gallerie
SN	Swiss Norm
SNCF	Société nationale des chemins de fer français
Strada-DB	Banque de données routières
SRL	Support Reaction Line
TDB	Tunnel Data Base
VSS	Schweizerischer Verband der Strassen- und Verkehrsfachleute
UVEK	Eidgenössisches Departement für Umwelt, Verkehr, Energie und Kommunikation

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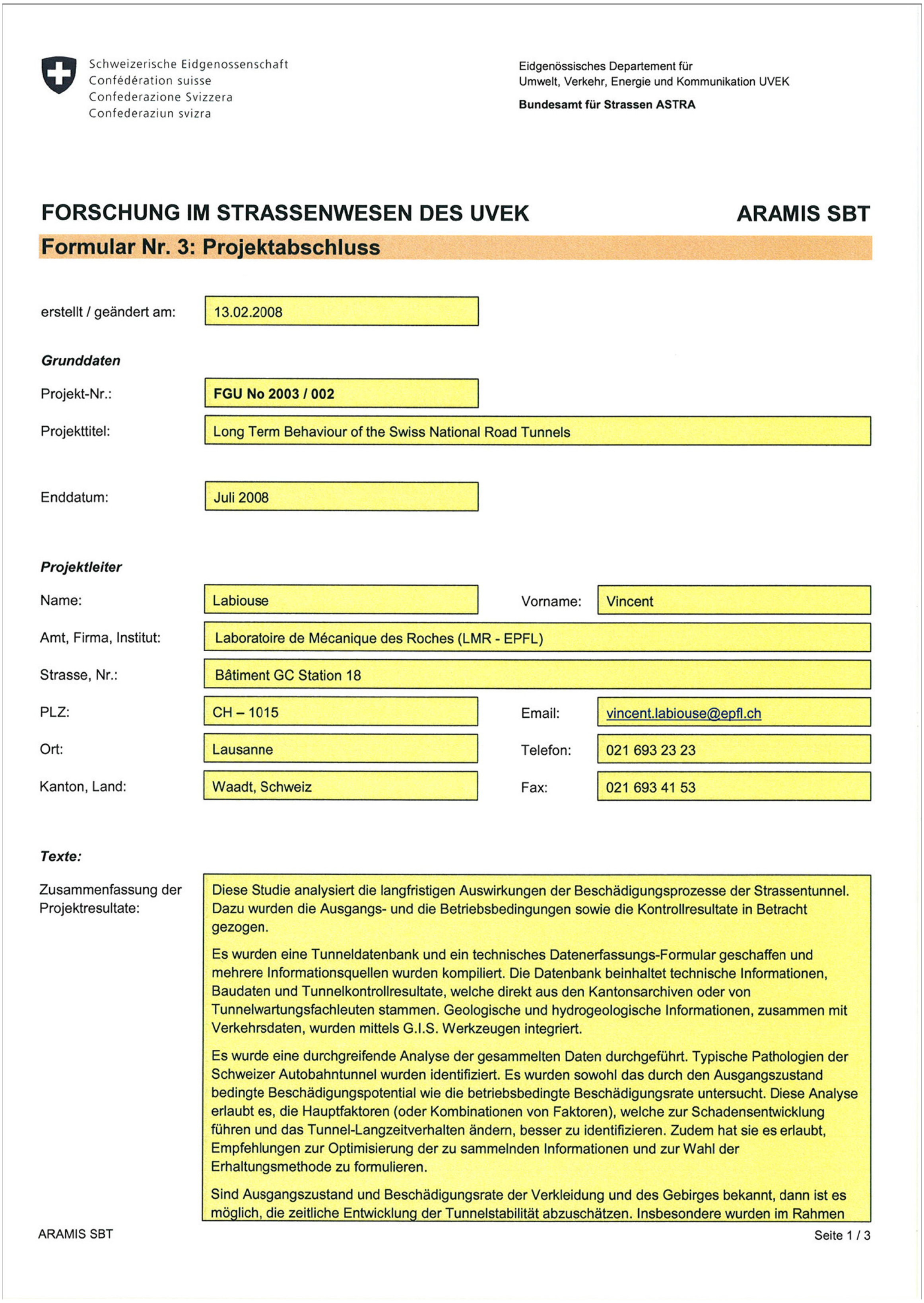
Project closure report



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Bundesamt für Strassen ASTRA

FORSCHUNG IM STRASSENWESEN DES UVEK **ARAMIS SBT**

Formular Nr. 3: Projektabschluss

erstellt / geändert am: 13.02.2008

Grunddaten

Projekt-Nr.:	**FGU No 2003 / 002**
Projekttitel:	Long Term Behaviour of the Swiss National Road Tunnels
Enddatum:	Juli 2008

Projektleiter

Name:	Labiouse	Vorname:	Vincent
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Texte:

Zusammenfassung der Projektresultate:

Diese Studie analysiert die langfristigen Auswirkungen der Beschädigungsprozesse der Strassentunnel. Dazu wurden die Ausgangs- und die Betriebsbedingungen sowie die Kontrollresultate in Betracht gezogen.

Es wurden eine Tunneldatenbank und ein technisches Datenerfassungs-Formular geschaffen und mehrere Informationsquellen wurden kompiliert. Die Datenbank beinhaltet technische Informationen, Baudaten und Tunnelkontrollresultate, welche direkt aus den Kantonsarchiven oder von Tunnelwartungsfachleuten stammen. Geologische und hydrogeologische Informationen, zusammen mit Verkehrsdaten, wurden mittels G.I.S. Werkzeugen integriert.

Es wurde eine durchgreifende Analyse der gesammelten Daten durchgeführt. Typische Pathologien der Schweizer Autobahntunnel wurden identifiziert. Es wurden sowohl das durch den Ausgangszustand bedingte Beschädigungspotential wie die betriebsbedingte Beschädigungsrate untersucht. Diese Analyse erlaubt es, die Hauptfaktoren (oder Kombinationen von Faktoren), welche zur Schadensentwicklung führen und das Tunnel-Langzeitverhalten ändern, besser zu identifizieren. Zudem hat sie es erlaubt, Empfehlungen zur Optimisierung der zu sammelnden Informationen und zur Wahl der Erhaltungsmethode zu formulieren.

Sind Ausgangszustand und Beschädigungsrate der Verkleidung und des Gebirges bekannt, dann ist es möglich, die zeitliche Entwicklung der Tunnelstabilität abzuschätzen. Insbesondere wurden im Rahmen

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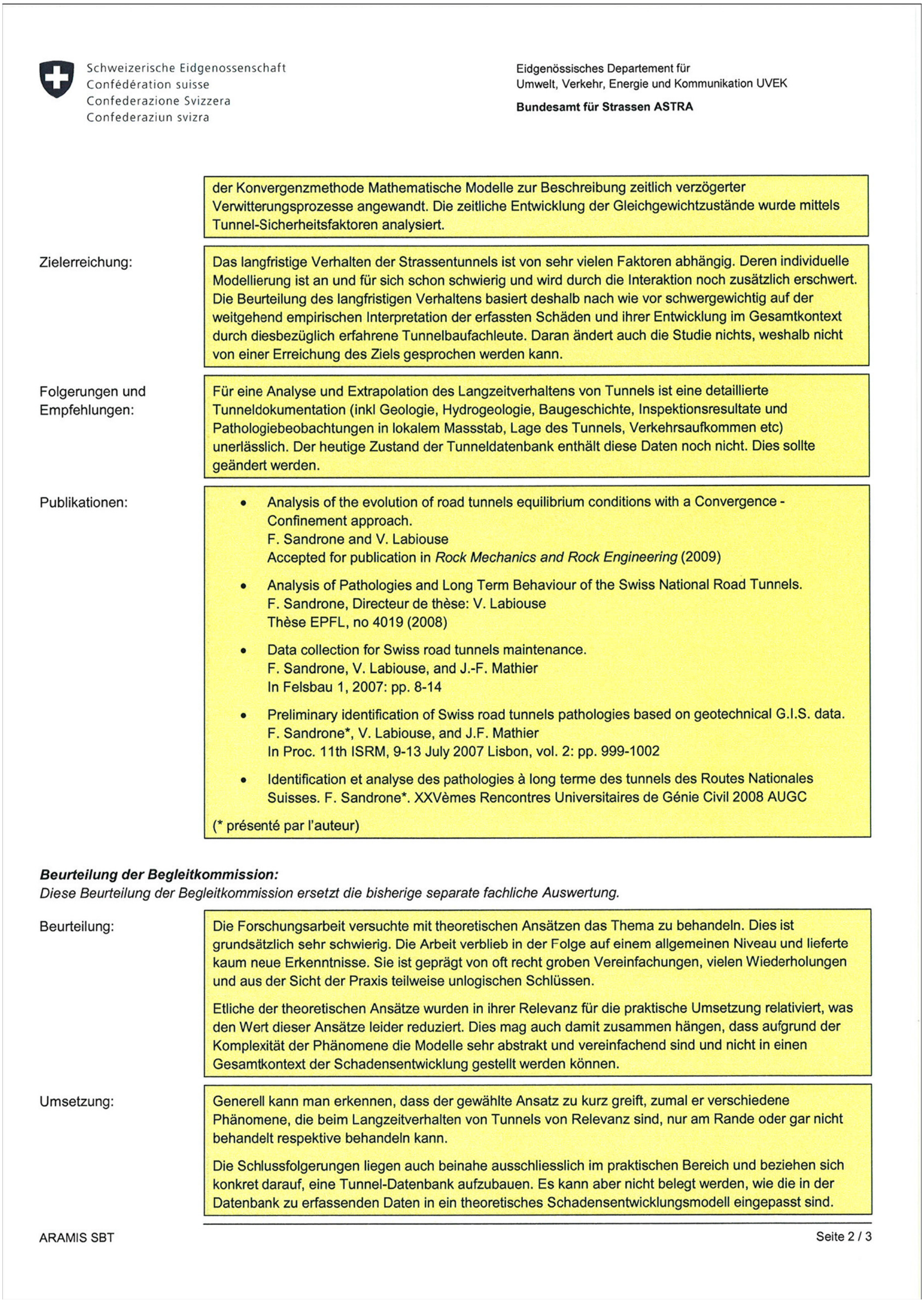

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der Konvergenzmethode Mathematische Modelle zur Beschreibung zeitlich verzögerter Verwitterungsprozesse angewandt. Die zeitliche Entwicklung der Gleichgewichtzustände wurde mittels Tunnel-Sicherheitsfaktoren analysiert.

Zielerreichung:

Das langfristige Verhalten der Strassentunnels ist von sehr vielen Faktoren abhängig. Deren individuelle Modellierung ist an und für sich schon schwierig und wird durch die Interaktion noch zusätzlich erschwert. Die Beurteilung des langfristigen Verhaltens basiert deshalb nach wie vor schwergewichtig auf der weitgehend empirischen Interpretation der erfassten Schäden und ihrer Entwicklung im Gesamtkontext durch diesbezüglich erfahrene Tunnelbaufachleute. Daran ändert auch die Studie nichts, weshalb nicht von einer Erreichung des Ziels gesprochen werden kann.

Folgerungen und Empfehlungen:

Für eine Analyse und Extrapolation des Langzeitverhaltens von Tunnels ist eine detaillierte Tunneldokumentation (inkl Geologie, Hydrogeologie, Baugeschichte, Inspektionsresultate und Pathologiebeobachtungen in lokalem Massstab, Lage des Tunnels, Verkehrsaufkommen etc) unerlässlich. Der heutige Zustand der Tunneldatenbank enthält diese Daten noch nicht. Dies sollte geändert werden.

Publikationen:

- Analysis of the evolution of road tunnels equilibrium conditions with a Convergence - Confinement approach.
  F. Sandrone and V. Labiouse
  Accepted for publication in *Rock Mechanics and Rock Engineering* (2009)
- Analysis of Pathologies and Long Term Behaviour of the Swiss National Road Tunnels.
  F. Sandrone, Directeur de thèse: V. Labiouse
  Thèse EPFL, no 4019 (2008)
- Data collection for Swiss road tunnels maintenance.
  F. Sandrone, V. Labiouse, and J.-F. Mathier
  In Felsbau 1, 2007: pp. 8-14
- Preliminary identification of Swiss road tunnels pathologies based on geotechnical G.I.S. data.
  F. Sandrone*, V. Labiouse, and J.F. Mathier
  In Proc. 11th ISRM, 9-13 July 2007 Lisbon, vol. 2: pp. 999-1002
- Identification et analyse des pathologies à long terme des tunnels des Routes Nationales Suisses. F. Sandrone*. XXVèmes Rencontres Universitaires de Génie Civil 2008 AUGC

(* présenté par l'auteur)

Beurteilung der Begleitkommission:
Diese Beurteilung der Begleitkommission ersetzt die bisherige separate fachliche Auswertung.

Beurteilung:

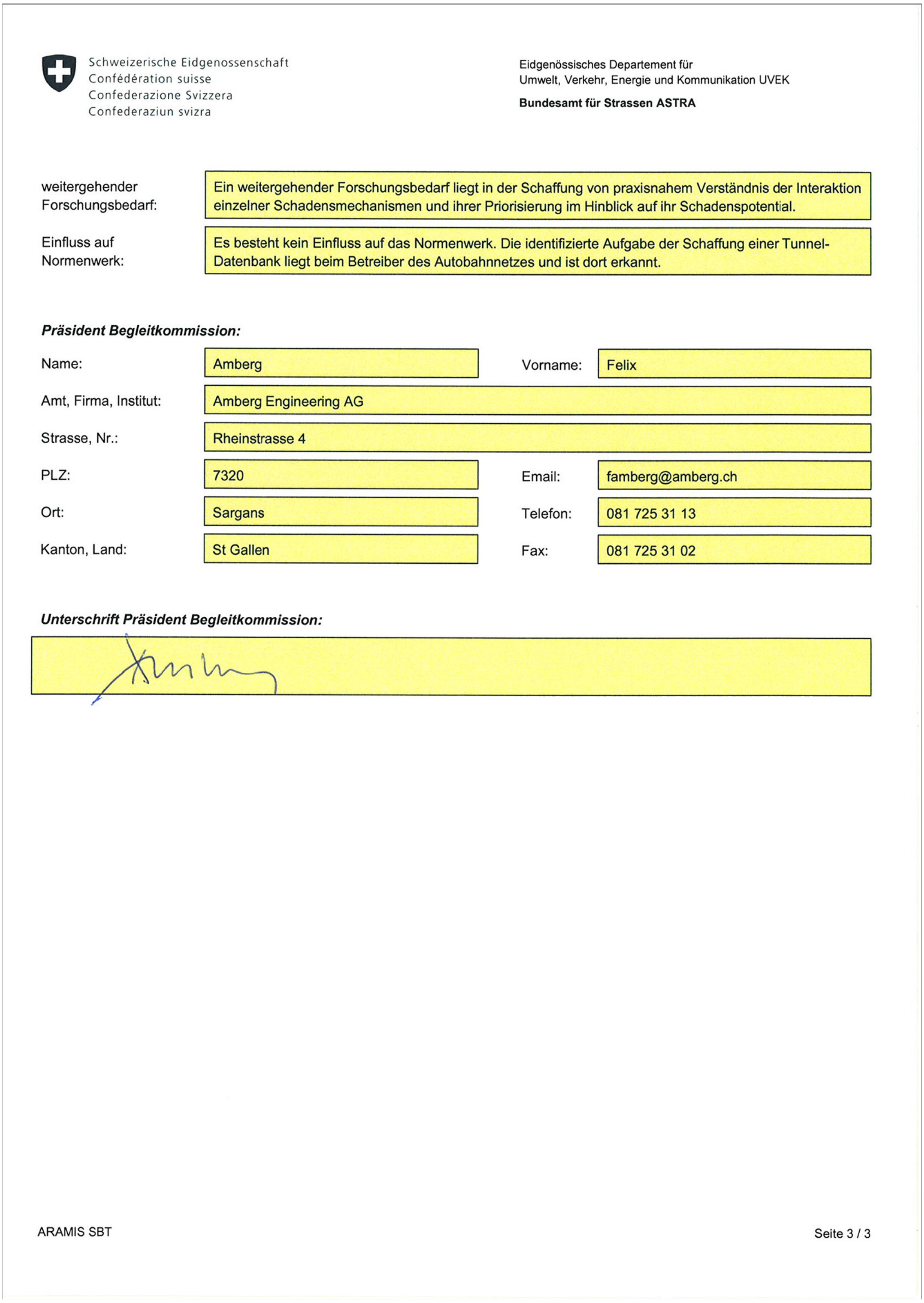
Die Forschungsarbeit versuchte mit theoretischen Ansätzen das Thema zu behandeln. Dies ist grundsätzlich sehr schwierig. Die Arbeit verblieb in der Folge auf einem allgemeinen Niveau und lieferte kaum neue Erkenntnisse. Sie ist geprägt von oft recht groben Vereinfachungen, vielen Wiederholungen und aus der Sicht der Praxis teilweise unlogischen Schlüssen.

Etliche der theoretischen Ansätze wurden in ihrer Relevanz für die praktische Umsetzung relativiert, was den Wert dieser Ansätze leider reduziert. Dies mag auch damit zusammen hängen, dass aufgrund der Komplexität der Phänomene die Modelle sehr abstrakt und vereinfachend sind und nicht in einen Gesamtkontext der Schadensentwicklung gestellt werden können.

Umsetzung:

Generell kann man erkennen, dass der gewählte Ansatz zu kurz greift, zumal er verschiedene Phänomene, die beim Langzeitverhalten von Tunnels von Relevanz sind, nur am Rande oder gar nicht behandelt respektive behandeln kann.

Die Schlussfolgerungen liegen auch beinahe ausschliesslich im praktischen Bereich und beziehen sich konkret darauf, eine Tunnel-Datenbank aufzubauen. Es kann aber nicht belegt werden, wie die in der Datenbank zu erfassenden Daten in ein theoretisches Schadensentwicklungsmodell eingepasst sind.



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weitergehender Forschungsbedarf: Ein weitergehender Forschungsbedarf liegt in der Schaffung von praxisnahem Verständnis der Interaktion einzelner Schadensmechanismen und ihrer Priorisierung im Hinblick auf ihr Schadenspotential.

Einfluss auf Normenwerk: Es besteht kein Einfluss auf das Normenwerk. Die identifizierte Aufgabe der Schaffung einer Tunnel-Datenbank liegt beim Betreiber des Autobahnnetzes und ist dort erkannt.

Präsident Begleitkommission:

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Amt, Firma, Institut:	Amberg Engineering AG		
Strasse, Nr.:	Rheinstrasse 4		
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Unterschrift Präsident Begleitkommission:

Index des rapports de recherche en matière de route

Bericht-Nr.	Projekt Nr.	Titel	Datum
1334	ASTRA 2009/009	"Was treibt uns an ? Antriebe und Treibstoffe für die Mobilität von Morgen Transports de l'avenir ? Moteurs et carburants pour la mobilité de demain What drives us on ? Drives and fuels for the mobility of tomorrow"	2011
1335	VSS 2007/502	"Stripping bei lärmmindernden Deckschichten unter Überrollbeanspruchung im labormasstab Désenrobage des enrobés peu bruyants des couches de roulement sous sollicitation de roulement en laboratoire Stripping of Low Noise Surface Courses during Laboratory Scaled Wheel Tracking"	2011
1336	ASTRA 2007/006	"SPIN-ALP: Scanning the Potential of Intermodal Transport on Alpine Corridors SPIN-ALP: Abschätzung des Potentials des Intermodalen Verkehrs auf Alpenkorridoren SPIN-ALP: Estimation du potentiel du transport intermodal sur les axes transalpins"	2010
1339	SVI 2005/001	"Widerstandsfunktionen für Innerorts- Strassenabschnitte ausserhalb des Einflussbereiches von Knoten Fonctions de résistance pour des tronçons routiers urbains en dehors de la zone d'influence de carrefours Capacity restraint functions for urban road sections not affected by intersection delays"	2010

Bericht-Nr.	**Projekt Nr.**	**Titel**	**Datum**
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. Die charakteristischen Indikatoren einer Velostadt. Evaluationsmethode der Velopolitiken anhand von 8 Indikatorgruppen für kleine und mittlere Gemeinden Characteristic indices of a Bike City. Method of evaluation of cycling policies in 8 indices for small and medium-sized communes"	2010
1337	ASTRA 2006/015	"Development of urban network travel time estimation methodology Temps de parcours en réseau urbain Methodologie für Fahrzeitbewertung in städtischen Strassennetz"	2011
1338	VSS 2006/902	"Wirkungsmodelle für fahrzeugseitige Einrichtungen zur Steigerung der Verkehrssicherheit Modèles d'impact d'équipements de véhicules pour améliorer la sécurité routière Modelling of the impact of in-vehicle equipment for the enhancement of traffic safety"	2009
1341	FGU 2007/005	"Design aids for the planning of TBM drives in squeezing ground Entscheidungsgrundlagen und Hilfsmittel für die Planung von TBM-Vortrieben in druckhaftem Gebirge Critères de décision et outils pour la planification de l'avancement au tunnelier dans des conditions de roches poussantes"	2011
1343	VSS 2009/903	"Basistechnologien für die intermodale Nutzungserfassung im Personenverkehr Basic technologies for detecting intermodal traveling passengers Les technologies de base pour l'enregistrement automatique des usagers de moyens de transports"	2011
1340	SVI 2004/051	"Aggressionen im Verkehr L'aggressivité au volant Aggressive Driving"	2011

Bericht-Nr.	Projekt Nr.	Titel	Datum
1344	VSS 2009/709	"Initialprojekt für das Forschungspaket ""Nutzensteigerung für die Anwender des SIS"" Projet initial pour le paquet de recherche ""Augmentation de l'utilité pour les usagers du système d'information de la route"" Initial project for the research package ""Increasing benefits for the users of the road and transport information system"""	2011
1345	SVI 2004/039	"Einsatzbereiche verschiedener Verkehrsmittel in Agglomerationen Application areas of various means of transportation in agglomerations Domaine d'application de different moyen de transport dans les agglomérations"	2011
1342	FGU 2005/003	"Untersuchungen zur Frostkörperbildung und Frosthebung beim Gefrierverfahren Investigations of the ice-wall grow and frost heave in artificial ground freezing Recherches sur la formation corps gelés et du soulèvement au gel pendant la procédure de congélation"	2010
647	AGB 2004/010	"Quality Control and Monitoring of electrically isolated post-tensioning tendons in bridges Qualitätsprüfung und Überwachung elektrisch isolierter Spannglieder in Brücken Contrôle de la qualité et surveillance des câbles de précontrainte isolés électriquement dans les ponts"	2011
1348	VSS 2008/801	"Sicherheit bei Parallelführung und Zusammentreffen von Strassen mit der Schiene Sécurité en cas de tracés rail-route parallèles ou rapprochés Safety measures to manage risk of roads meeting or running close to railways"	2011
1349	VSS 2003/205	"In-Situ-Abflussversuche zur Untersuchung der Entwässerung von Autobahnen On-site runoff experiments on roads Essai d'écoulements pour l'évacuation des eaux des autoroutes"	2011

Bericht-Nr.	Projekt Nr.	Titel	Datum
1350	VSS 2007/904	"IT-Security im Bereich Verkehrstelematik IT-Security pour la télématique des transports IT-Security for Transport and Telematics"	2011
1352	VSS 2008/302	"Fussgängerstreifen (Grundlagen) Passage pour piétons (les bases) Pedestrian crossing (basics)"	2011
1346	ASTRA 2007/004	"Quantifizierung von Leckagen in Abluftkanälen bei Strassentunneln mit konzentrierter Rauchabsaugung Quantification of the leakages into exhaust ducts in road tunnels with concentrated exhaust systems Quantification des fuites des canaux d'extraction dans des tunnels routiers à extraction concentrée de fumée"	2010
1351	ASTRA 2009/001	"Development of a best practice methodology for risk assessment in road tunnels Entwicklung einer besten Praxis-Methode zur Risikomodellierung für Strassentunnelanlagen Développement d'une méthode de meilleures pratiques pour l'analyse des risques dans les tunnels routiers"	2011
1355	FGU 2007/002	"Prüfung des Sulfatwiderstandes von Beton nach SIA 262/1, Anhand D: Anwendbarkeit und Relevanz für die Praxis Essai de résistance aux sulfates selon la norme SIA 262/1, Annexe D: Applicabilité et importance pour la pratique Testing sulfate resistance of concrete according to SIA 262/1, appendix D: applicability and relevance for use in practice"	2011
1356	SVI 2007/014	"Kooperation an Bahnhöfen und Haltestellen Coopération dans les gares et arrêts Coopération at railway stations and stops"	2011
1362	SVI 2004/012	Aktivitätenorientierte Analyse des Neuverkehrs Activity oriented analysis of induced travel demand Analyse orientée aux acitivtés du trafic induit	2012
1361	SVI 2004/043	"Innovative Ansätze der Parkraukmbewirtschaftung Approches innovantes de la gestion du stationnement Innovative approaches to parking management"	2012

Bericht-Nr.	**Projekt Nr.**	**Titel**	**Datum**
1357	SVI 2007/007	"Unaufmerksamkeit und Ablenkung: Was macht der Mensch am Steuer? Driver Inattention and Distraction as Cause of Accident: How do Drivers Behave in Cars? L'inattention et la distraction: comment se comportent les gens au volant?"	2012