

Chapter 2

Environmental-Geological Problems due to Underground Works

2.1 Introduction

Considering the environmental impact, the realization of underground works usually implies some advantages, such as the availability of free space on the surface, lower visual and noise impact and little effects on ecosystems (Barla and Barla 2002). But sometimes there are disadvantages as well, such as higher construction and maintenance costs, the alteration of complex natural balances, a local increase in noise and vibrations in the construction and operating phases. Actually, underground works are usually more expensive, but the cost and benefit balance is positive nevertheless if one considers the social-economical value of the area used, in particular, in urban areas or areas with beautiful or protected landscapes and ecosystems. Obviously, the negative aspects of the infrastructure are not cancelled, still the more evident ones are removed or reduced and others are produced on different elements, subjects and temporal scales (Table 2.1).

The main environmental problems linked to the construction of underground works are listed below:

- Triggering of surface settlements, structures collapses and slope instabilities
- Drying up of springs and groundwater alterations
- Storage and use of excavated materials
- Noise
- Vibrations
- Pollution of groundwater, mainly after the realization of stabilization works by injections
- Emission of dust and air pollutants

As far as air pollution is concerned, apart from the obvious increase of pollutants in the construction phase, in theory a road tunnel is built to reduce traffic, noise and pollutant emissions, in particular in urban areas. Actually, the cost–advantage balance also has to take into account a local increase in those pollutants (in particular, carbon monoxide, nitrogen oxides, suspended solids, dusts and benzene) in the area of a few hundred metres surrounding the tunnel portal. Those aspects, as well as

Table 2.1 Explanatory confrontation among the effects of road and railway tunnels compared to corresponding infrastructures on the surface

		Construction	Operating phase	Maintenance
Environmental aspects	Air	+	+	
	Geomorphology	+		
	Groundwater	—		
	Surface waters	+		
	Ground	+		
	External noise	+	+	+
	Inside noise	—	—	—
	Vibrations	—	—	—
	Vegetation	+		+
	Fauna	+	+	+
	Landscape	+	+	+
	Surface ground heritage	+		
	Underground heritage	—		
	Soil use	+	+	+
	Infrastructures/facilities	+	+	
	Safety	—	+	
	Efficiency		+	
	Human psychology		—	
Economic aspects	Costs	—	—	—

+ environmental aspects determining the advantages of an underground work compared to the same work on the surface

— environmental aspects determining the disadvantages of an underground work compared to the same work on the surface

groundwater pollution, are not discussed in this book; this book is focused on the analysis of the more typical geological and hydrogeological problems.

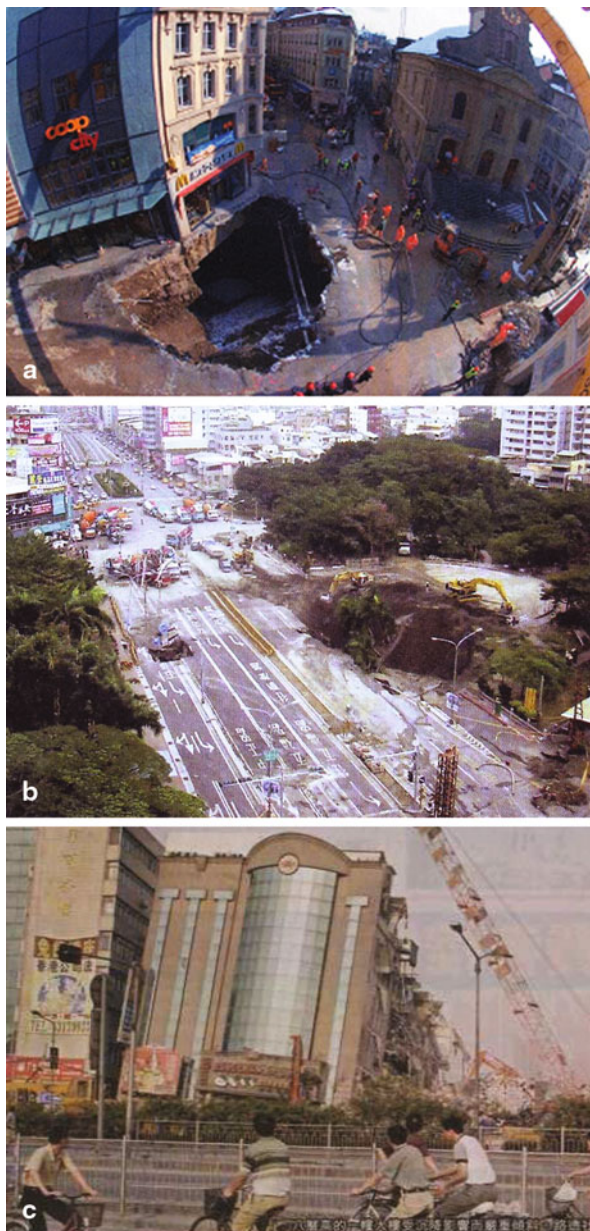
2.2 Surface Settlements

The opening of underground works causes a deformation of the soils and rocks around the excavation area. Such deformations may trigger sudden collapses, subsidence and sinking that can damage both the work under construction and pre-existing nearby structures, in particular, if the work is being constructed in developed areas (Fig. 2.1).

The consequences and the damages depend on the intensity of the phenomenon and on the vulnerability of the elements on the surface (buildings, rivers, industrial settlements, facilities etc.). Generally, the response to the opening of the underground work and, as a consequence, the extent of settlements depends on the following elements:

- Excavation technique
- Dimension and geometry of the excavation
- Type of excavated material

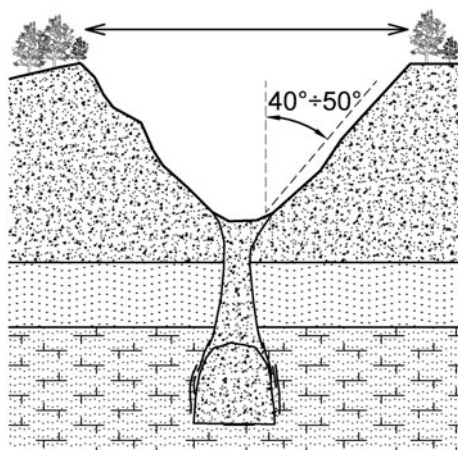
Fig. 2.1 Examples of the formation of crown holes and surface settlements induced by tunnelling: **a** Crater of Saint-Laurent Place (Metro Lausanne-Ouchy; Seidenfuß 2006). **b** Kaohsiung collapses (from T&T International 2006). **c** Shanghai Metro (P.R. of China), 2003. (Wannick 2006)



The sudden collapse with cavity filling (Fig. 2.2) usually happens in case of limited overburden that can reach, in some particular cases, up to ten times the cavity diameter.

In case of subsidence phenomena, the deformation can be delayed with respect to the excavation phase and can involve wider areas. In this case, the typical parameters

Fig. 2.2 Mechanism of sudden collapse with cavity filling



are the shifts, mainly vertical and variable on the surface from point to point, and the width of the area interested by subsidence. Usually, this is included in a dihedral angle whose opening varies between 15° and 45° according to the depth of the excavation and the features of the soil.

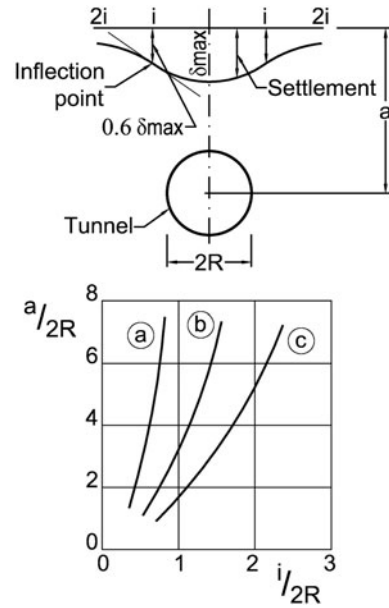
A generalized and uniform sinking does not cause big damages, whereas discrete settlements may seriously damage overlying structures and facilities.

Usually, the forecast of surface movement associated with the excavation, when no structures are present on the ground level, is carried out by means of empiric relations based on the following hypothesis: In a section transverse to the tunnel axis and at an adequate distance from the excavation face, the outline of the collapses can be approximated to an inverted Gaussian distribution (Fig. 2.3). In order to characterize the width of the area interested by deformations, Peck (1969) proposed to interpolate the deformed area of the topographic surface with a Gaussian curve and to take the horizontal measures between the tunnel axis and the inflection point of the curve as characteristic dimension.

Data on the behaviour of models in 1:1 scale show that the relation $i/2R$ (R being the tunnel radius) increases when $a/2R$ increases (a being the tunnel depth from land surface) according to different laws in different soils.

Nevertheless, it is well known that the presence of a building or a structure may change the subsidence profile. Numerical model can be used if the possible impact of an underground work on pre-existing structures must be evaluated. Possibly, the modelling should be carried out in a three-dimensional field to be able to correctly simulate the different relative tunnel-structure positions, which means the study should not be limited to the case of the tunnel axes perpendicular to the main level of the structure. Obviously, the reliability of the results of the numerical modelling depends on the quality of input data of the model, in particular those concerning the deformation parameters of the ground material and its heterogeneity (Gattinoni et al. 2012).

Fig. 2.3 Surface settlements caused by the construction of a tunnel with radius R situated at shallow depth, in the presence of: **a** rocks, hard clays, sands over the groundwater table, **b** soft to hard clays and **c** sand under the groundwater table



In case the load change is due to a lowering of the groundwater table, the subsidence phenomenon is ruled by the following equations:

$$S_{\infty} = \Delta q \cdot \frac{H}{E} \quad \text{with: } \Delta q = \sigma'_{\infty} - \sigma'_{in}$$

where:

s_{∞} final settlement

Δq change of the applied load

H thickness of the soil interested by the load change

E deformation modulus of the material

σ'_{in} initial effective stress

σ'_{∞} final effective stress

According to a first estimate, the effective stress at the depth of the underground work, before its construction, can be calculated as follows:

$$\begin{aligned} \sigma'_{in} &= \sigma_{in} - u_{in} \\ \sigma_{in} &= \gamma_d \cdot h_t - \gamma_d \cdot h_w + \gamma_{sat} \cdot h_w = \gamma_d(h_t - h_w) + \gamma_{sat} \cdot h_w \\ u_{in} &= \gamma_w \cdot h_w \end{aligned}$$

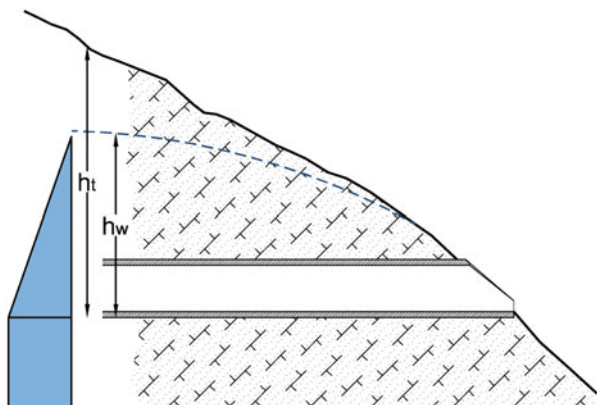
where:

γ_s specific weight of the grains

γ_d specific weight of dry material

γ_{sat} specific weight of saturated material

Fig. 2.4 Load change induced by the groundwater table drawdown caused by the opening of an underground work



γ_w specific weight of water

m porosity of the material

h_t excavation depth with regards to land surface

h_w piezometric height above the excavation level

If a total drainage of the soil up to the excavation level is considered, the effective stress after excavation becomes:

$$\sigma_{\infty} = \gamma_s \cdot (1 - m) \cdot h_t = \gamma_d \cdot h_t$$

$$u_{\infty} = 0$$

$$\sigma'_{\infty} = \sigma_{\infty} - u_{\infty} = \sigma_{\infty}$$

It is therefore possible to calculate the maximum load change at the level of the underground work and to consider a decreasing trend toward the surface (Fig. 2.4).

2.3 Slope Instability

The interaction with slope stability is a typical problem of the portal stretches and underground works carried out close to the side of a slope (Figs. 2.5–2.7). The excavation of an underground work implies the annihilation of the stress state in correspondence of its boundaries, the redistribution of stresses with local increases of the deviatoric stresses. In terms of stress state, the global effect depends on the following elements:

- Characteristics of the excavation (site, shape and dimension)
- Excavation technique
- Material constitutive laws
- Initial stress state (including the water neutral pressures)

Figure 2.8 shows an example of change in the slope stability conditions after the construction of a tunnel close to the side of the slope, according to its depth. In

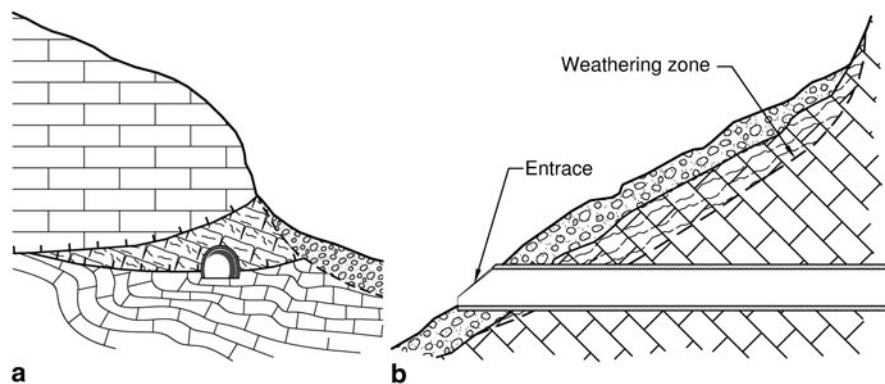


Fig. 2.5 Examples of a geological structure potentially interested by excavation interactions with the slope stability: **a** Overthrust zone, **b** portal stretches

Fig. 2.6 Tunnel portal affecting the slope obliquely: stabilization with tie rods. (By Pizzarotti)



that case, the analysis was carried out with limit equilibrium techniques in a two-dimensional field, and it refers to dry conditions. The problem is complex and should be studied in a three-dimensional field considering the tunnel progress. Furthermore, when dealing with materials having small grain size, the stability analysis should be referred to three different phases:

- Initial (short term)
- Transient
- Final (long term)

In practical terms, the assessment of slope stability conditions in relations to tunnelling can be assimilated to that of any slope. Therefore, specific books can be consulted for its detailed dissertation.



Fig. 2.7 Tunnel portal affecting the slope obliquely: in this case, stabilization structures were: **a** placement of tie rods on a reinforced concrete head-beam and a steel ribs-reinforced portal, **b** completion of the reinforced concrete structure. (By Pizzarotti)

As far as slope stability is concerned, another quite typical problem in tunnelling is related to the presence of deep-seated gravitational deformation (DSGD), which is a common and widespread type of large slope instability in the Alps (Agliardi et al. 2013). The presence of DSGD assumes a great importance both during the tunnel construction and during its lifetime, as it is often correlated with important cataclastic systems. Actually, DSGDs can affect the tunnel-boring machine (TBM) parameters and then the excavation speed, just like Pont Ventoux hydroelectric power plan tunnel (Venturini et al. 2001). They also have a very important impact on groundwater flow and tunnel inflow (Vincenzi et al. 2010). Moreover, the progressive deformation of DSGD can lead to important deformative phenomena within a tunnel if its lining is not properly designed, by taking into account the stresses related to DSGD. This is the case of the Mt. Piazza tunnel (Northern Italy), which cuts a metamorphic mica schist formation (“Scisti dei Laghi” Auct.): The tunnel suffers from the M. Legnoncino DSGD, which causes deformations in the order of several millimetres in the cladding, requiring constant maintenance; hence, the tunnel was recently closed as a consequence of the increasing deformations.

The geognostic investigations useful to detect and characterize DSGDs consist of a detailed morpho-structural and morpho-dynamic analysis (Cadoppi et al. 2007), remote sensing (Strozzi et al. 2013) and geophysical system (Supper et al. 2013).

2.4 Interaction with Surface Water and Groundwater

The interaction between tunnelling and groundwater is a very relevant problem not only due to the need to safeguard water resources from impoverishment and pollution risk, but also to guarantee the safety of workers and the effectiveness of the draining works. One of the most emblematic examples (Table 2.2) in Europe concerns the construction of the Gran Sasso (Italy) motorway tunnels, that were interested by water inflows exceeding 2,000 l/s, and the railway tunnel for the Bologna–Firenze (Italy) high-speed stretch (Rossi et al. 2001), with drained flows reaching 650 l/s.

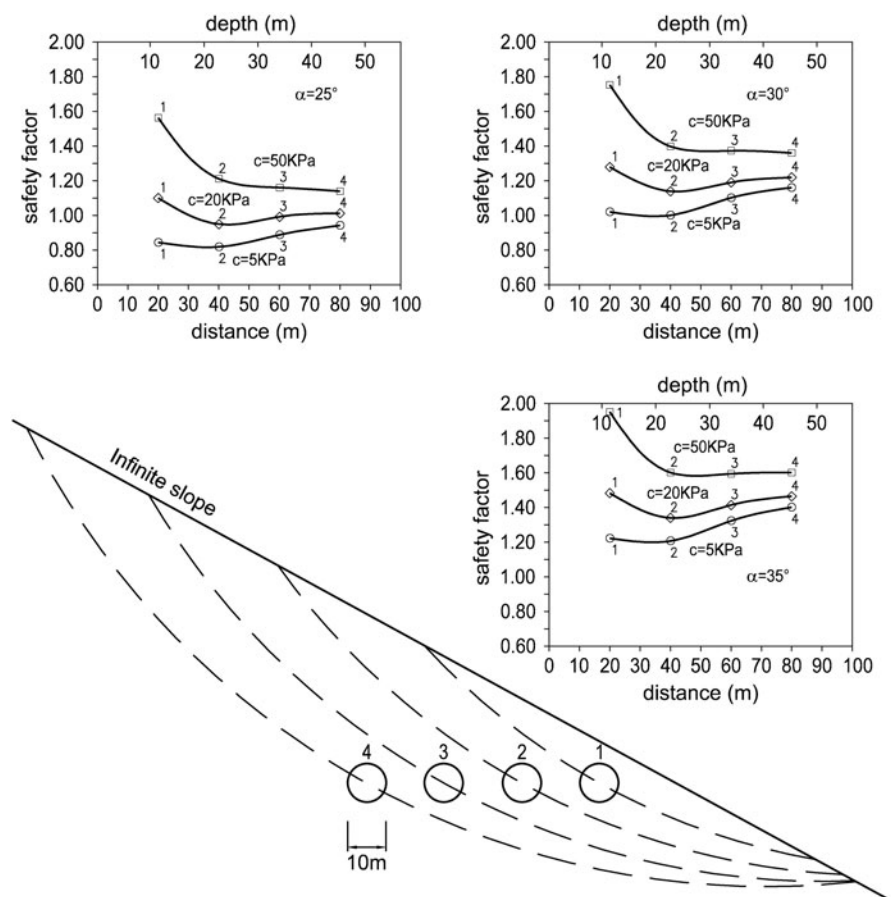


Fig. 2.8 Example of change in a slope stability after the construction of a tunnel near the side of the slope, according to its depth. (Picarelli et al. 2002)

On the contrary, when the tunnels in San Pellegrino Terme (Bergamo–Italy) were constructed, the interdisciplinary approach of the study and the attention to the environment guaranteed the safeguard of the hydrothermal springs of the area (Barla 2000).

It is well known that the excavation of tunnels has a relevant draining effect leading to a more or less generalized drawdown of the groundwater table (Figs. 2.9 and 2.10), whose effects may be undesirable, such as: the drying up of springs (Figs. 2.10 and 2.11) and/or wells, qualitative changes of the groundwater, changes in the vegetations, changes in the slope stability, changes in the flow and in the quality of thermal waters, changes of the hydrogeological balance at the basin scale. These phenomena can persist even after the tunnel construction if the final alignment is not completely waterproof.

Table 2.2 Examples of tunnel inflow during excavation. (Modified from Civita et al. 2002)

Tunnel	Type	<i>L.</i> (km)	$Q_{\max}$ (m ³ /s)	$Q_{\min}$ (m ³ /s)	Aquifer
Sempione (ITA–CH)	Railway	19.8	1.700	0.864	Limestone
Vaglia (BO–FI)	Railway	18.6	0.080	–	Limestone, calcarenite, sandstone
Direttissima (BO–FI)	Railway	18.5	1.200	0.060	Sandstone
Pavoncelli bis (AV)	Hydraulic	15.5	0.800	0.070	Limestone, clay
Firenzuola (BO–FI)	Railway	15.1	0.277	0.070	Sandstone and marl
Santomarco (Paola–CS)	Railway	15.3	0.100	0.038	Metamorphites
Frejus (T4)	Highway	12.9	0.007	0.001	Several
M. Bianco (Ti)	Highway	11.6	0.800	0.440	Granite
Raticosa (BO–FI)	Railway	10.4	0.037	–	Sandstone, marl and clay
Gran Sasso (A24)	Highway	10.2	3.000	0.600	Limestone
S. Lucia (NA–SA)	Railway	10.2	1.000	0.250	Limestone
Putifigari (SS)	Road	9.8	0.070	0.050	Vulcanites
Zuc del Bor (UD–AUT)	Railway	9.3	0.700	0.650	Limestone
S. Stefano (GE–F)	Railway	7.9	–	high	Marly limestone, sandstone
M. Olimpino 2 (MI–CO)	Railway	7.2	very high	–	Limestone, sands
Serena (PR–SP)	Railway	6.9	medium	–	Calcarenites, breccia, flysch
M. La Mula	Hydraulic	6.3	0.200	0.800	Limestone, dolomite
Turchino (GE–AT)	Railway	6.4	0.110	0.075	Calceschysts
Satriano (1° salto)	Hydraulic	6.4	very high	–	Milonitic granite
Gran S. Bernardo (T2)	Highway	5.9	low	very low	Gneiss, schist
S. Leopoldo (UD–AUT)	Railway	5.7	3.600	high	Limestone
Gravere (TO–FRA)	Railway	5.6	very high	0.013	Calceschysts
Vado Ligure (ITA–FRA)	Railway	4.9	0.200	0.050	Dolomite
Colle Croce (ITA–FRA)	Road	4.1	low	very low	Calceschysts
Col di Tenda (ITA–FRA)	Railway	3.2	0.600	0.200	Limestone
Bypass Spriana	Hydraulic	3.2	0.300	0.040	Gneiss, limestone, dolomite
Villeneuve (A5)	Highway	3.2	0.200	0.001	Calceschysts
Prè Saint Didier (A5)	Highway	2.8	0.100	0.080	Calceschysts, sandstone
Moro (AN–BA)	Railway	1.9	0.080	–	Gravels and sands
Colle della Scala	Railway	–	very high	high	Limestone
Crocchetta (Paola–CS)	Road	1.5	0.022	0.028	Tectonised schists

<http://www.springer.com/978-94-007-7849-8>

Engineering Geology for Underground Works

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2014, XIII, 305 p. 291 illus., 115 illus. in color.,

Hardcover

ISBN: 978-94-007-7849-8