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GEOMECHANIK

# **Geotechnical Monitoring in Conventional Tunnelling**

**Handbook**

**2014**





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## 1. PREFACE

Even with an excellent geological and geotechnical investigation the ground model developed during design contains uncertainties. These uncertainties and the simplifications made during design lead to a residual risk during construction. To achieve safe and economical construction in spite of the uncertainties, specific procedures, such as the observational approach within a geotechnical safety management plan have to be applied. Aspects of geotechnical design for underground structures are addressed in the OeGG guideline [1]. A general overview on monitoring of underground structures is given in the BTS guide [2].

Successful application of the observational approach requires sound preparation in the design phase and provisions accompanying the construction. To fully utilise the benefits of monitoring, all preparatory steps, like the establishment of the monitoring plan, contractual and organisational provisions, must be established prior to construction. Systematic and frequent monitoring, as well as observation and interpretation are important components of the observational method. The systematic monitoring of the system behaviour allows an optimisation of the construction and support concept, thus reducing the construction costs and vastly minimising the risks. The findings of the monitoring can be used to adjust the design of both the primary and secondary supports.

A program for monitoring, observation and interpretation is developed to capture the range of expected system behaviours. One of the main purposes of the program is revealing if the observed system behaviour lies within expected limits, and to validate the appropriateness of the excavation and support measures. Another aspect is the prediction of the system behaviour and the updating of the geological/geotechnical model.

In all cases the primary objective of monitoring is the assessment of the system stability and the impact on the environment. Depending on the location of the project and the possible consequences of a failure, the observation and evaluation methods may vary. For example in a tunnel with low overburden in built up areas, lining failures could lead to a collapse with serious damage to utilities and surface structures. As linings generally fail in a brittle manner without significant prior increase in the displacements, the continuous evaluation of the lining utilisation is of great importance.

Conversely in tunnels with high overburden, failure of the lining does not necessarily jeopardise the stability of the system, but may result only in additional displacements and costs. In this case the observation of the system stability and development of displacements in this case is the main objective of monitoring.

In addition the results of monitoring can be used for back analysing ground parameters.

Monitoring and observation rely on various sources of information, such as geological documentation and exploration, visual inspections of face and lining, as well as geotechnical measurements. Combined information from various sources facilitates assessment of the ground response, ground-support interaction (system behaviour), and three dimensional ground structure.

This handbook addresses all parties involved in the design and construction of tunnels. It is intended to show the state of the art in tunnel monitoring, data evaluation and interpretation.

It should assist in developing a monitoring program, execution of measurements, and enable maximum use of the acquired data by applying appropriate data evaluation and interpretation methods. It also contains typical monitoring layouts and requirements regarding reading accuracy.

The handbook presents a general concept for practical application of geotechnical monitoring in tunnelling. The examples included are intended for illustration purposes only, as monitoring programs and methods for evaluation and interpretation must be designed on a project specific basis.

## 2. DEFINITIONS

Design update	Update of the tunnel design based on additional information regarding the ground properties and system behaviour
Expected system behaviour	Range of system behaviour identified during tunnel design, considering the respective scatter of the ground properties. It fulfils the requirements of ultimate limit state, serviceability limit state and project specific requirements
Geotechnical safety management plan	Document containing monitoring program, definition of safety relevant issues, warning and alarm criteria, mitigation measures and action plan, definition of responsibilities and information and reporting structure, as well as contact information of all stakeholders
Ground behaviour	Reaction of the ground to the excavation of the full profile without the consideration of sequential excavation and support
Ground condition	Condition of the ground in terms of composition, stiffness, strength, structure, hydrology, etc.
Ground displacements	Displacements of soil or rock surrounding an underground structure
Ground type	Ground with similar properties
Mitigation measures	Measures in order to reduce the probability of an unfavourable event, e.g. increase of support, supplementary measures, etc.
Monitoring	Measurement of physical parameters
Monitoring data	Values obtained by measuring certain changes of the system behaviour, e.g. displacements, inclinations, strains, water level, pressure, etc.
Monitoring program	Compilation of means and methods for monitoring the system behaviour
Observation	Active acquisition of qualitative and quantitative information

Observational method	Continuous review of the behaviour and update of the design and adjustment of construction method during construction, based on actual conditions and observations, as required. Tender design is based on the assessment of the most probable conditions and likely variations
Observation variables	Variables with special relevance to the system behaviour
Resolution	The fineness of detail that can be distinguished in a measurement
Surface settlements	Vertical displacements of the ground surface
System accuracy	Resulting accuracy from instrumentation and evaluation
System behaviour	Behaviour resulting from the interaction between ground, excavation, and support
System stability	Stability of both ground and support
Warning and alarm criteria	Criteria which trigger a certain action

### 3. MONITORING PROJECT

#### 3.1. Workflow

During a geotechnical monitoring project, all items and courses of action, as depicted in Figure 1, should be considered. A concept needs to be developed on how to measure the key observation variables of the geotechnical problem under consideration. This includes selection of instrumentation type, monitoring layout, frequency of readings, data transmission and the associated evaluation method. The different work stages, as presented in Figure 1, are not explicitly discussed here. The associated issues are treated in the following chapters.

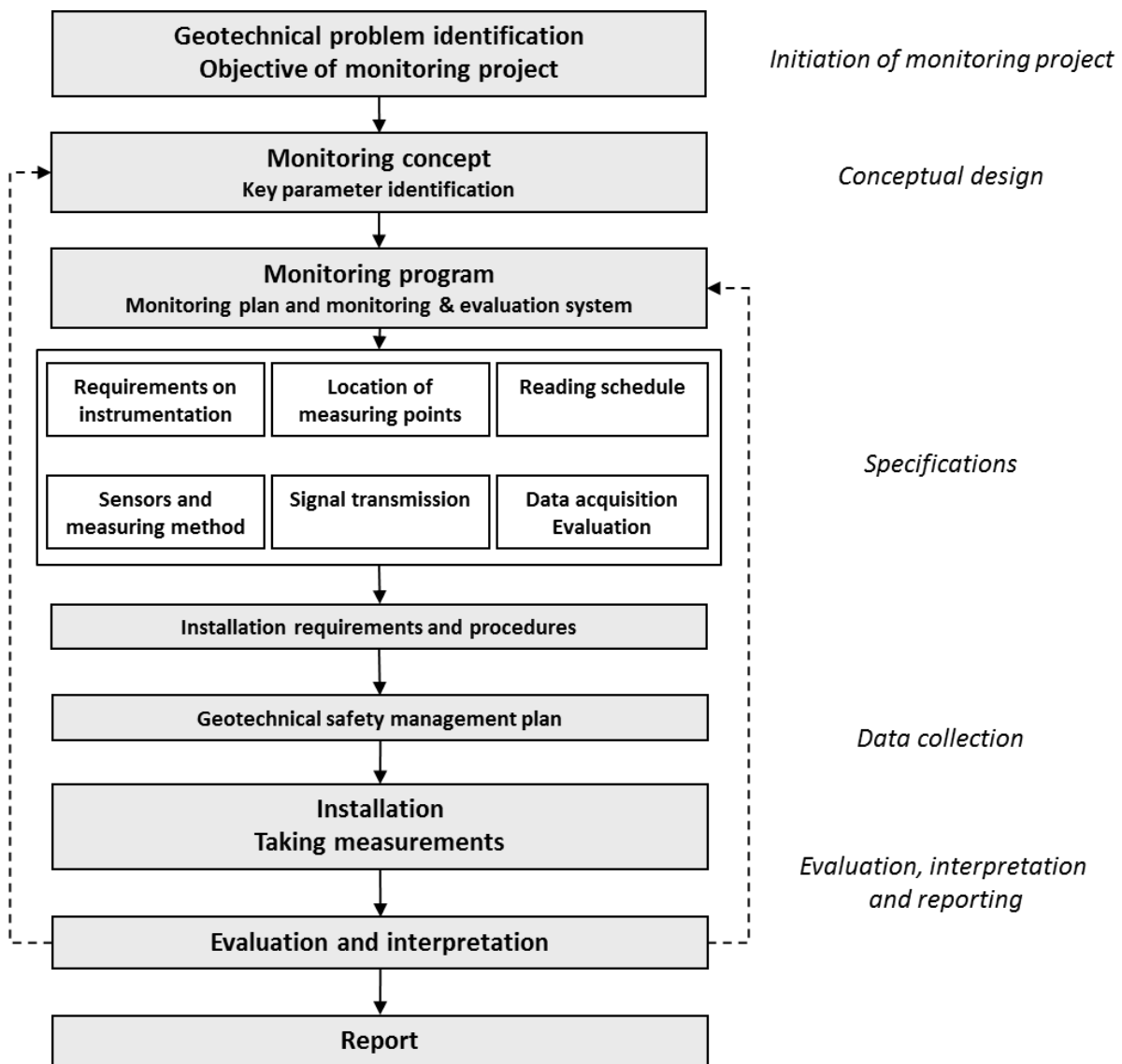


Figure 1. Basic concept of a geotechnical monitoring project (modified after preEN-ISO 18674 [3])

### **3.2. Definition of expected system behaviour**

The designer defines the expected system behaviour resulting from tunnelling, and its impacts on the surface, before construction commences. The following information must be available:

- Range of expected tunnel displacements for all construction phases
- Level of loading of the shotcrete lining in all construction phases
- Surface settlements and their spatial distribution
- Ground deformation in the area of structures
- Allowable distortion or curvature of installations and structures in the expected influence area of the underground construction, e.g. buildings, railway tracks, water pipes, wastewater sewers, gas pipes

### **3.3. Monitoring concept and identification of key observation variables**

The arrangement and layout of the various monitoring installations should be chosen under consideration of the specific project requirements and boundary conditions. The type, number, or spacing of the monitoring installations should consider the construction layout and sequence, the quality of the encountered ground and the surface situation in terms of sensitive structures (e.g. buildings, infrastructural utilities, etc.).

Depending on potential hazards, corresponding observation variables shall be defined (Table 1).

Table 1. Typical hazards and the corresponding observation variables

Hazard	Due to	Observation Variables						
		Surface settlements	Tilt angle	Support displacement / strains / utilisation / state	Water table / pressure	Face displacements	Ground structure	Ground displacements
Unacceptable surface settlements, displacements of structures and utilities	Consolidation	■	-	■	■	-	-	■
	Shear failure of the ground	■	□	■	-	-	-	■
	Excessive face deformation	■	□	-	-	■	-	-
	Invert failure	-	-	■	-	-	-	-
	Support damage and/or failure	-	-	■	-	-	-	-
Discontinuity induced detachment of blocks	Unfavourable discontinuity orientation	-	-	□	-	-	■	-
Clearance violation and/or lining failure	Excessive deformations	-	-	■	-	-	-	■
	Shear failure along dominant geological structures*	-	-	■	-	-	■	■
	Swelling ground	-	-	■	■	-	-	■
■ very valuable □ limited value - no value * - is also associated with the occurrence of excessive deformations, however asymmetric deformations along the systematic weakness planes predominantly influence the system behaviour.								

Table 2 presents the observation variables in relation to the applicable observation methods.

Table 2. Observation variables and applicable observation methods

	Absolute 3D displacement monitoring	Face displacement monitoring	Levelling	Extensometer	Load cell	Tilt meter	Hydrostatic levelling system	Borehole inclinometers	Piezometers	Water level gauge	Strain gauges	Invert probe	Geological compass-clinometer	Digital ground mapping	Visual inspection
Surface Settlements	■	-	■	■	-	-	□	■	-	-	-	-	-	-	-
Structures and utilities deformation	■	-	■	■	-	■	□	■	-	-	-	-	-	-	-
Lining displacements	■	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Lining strains	■*	-	-	-	-	-	-	-	-	-	■	-	-	-	-
Anchor loads	-	-	-	-	■	-	-	-	-	-	-	-	-	-	-
Invert integrity	■	-	-	■	-	-	-	-	-	-	■	■	-	-	■
Water level	-	-	-	-	-	-	-	-	■	■	-	-	-	-	-
Pore pressure	-	-	-	-	-	-	-	-	■	-	-	-	-	-	-
Face displacements	-	■	-	-	-	-	-	-	-	-	-	-	-	-	-
Ground structure	-	-	-	-	-	-	-	-	-	-	-	-	■	■	■
Ground displacements	-	-	-	■	-	-	-	■	-	-	-	-	-	-	-
Qualitative inspection of lining integrity	-	-	-	-	-	-	-	-	-	-	-	-	-	■	■
■ very valuable □ limited value - no value * - possible with computer-based interpolation and evaluation methods															

### **3.4. Organisation and management of the monitoring project**

- The execution of the monitoring, evaluation and analysis of the data should be preferably executed by the party carrying the geological risk
- Prediction and assessment of the system behaviour are essential elements of the detail design and thus should be executed by the designer
- Development of the monitoring plan and its update is responsibility of the designer
- Evaluation and analysis of the monitoring data, as well as of all other relevant information have to be done without delay by geotechnically qualified personnel, and the results made available to all parties involved as soon as possible
- The processed monitoring data and other relevant information shall be stored on a powerful IT device, and made available to all parties involved in the project
- Technical and organizational provisions should guarantee that data evaluation is rapid enough to allow for successful implementation of mitigation measures
- All steps of the process (measurement, evaluation, interpretation, recommendation of mitigation measures, decision, supervision of execution) shall be documented

### **3.5. Monitoring program**

The monitoring program includes the specification of the measuring procedure, the location of the monitoring devices and the monitoring schedule. Attention shall be paid to the fact that the monitoring results are often affected by instrumentation, installation and environmental effects.

The chosen type of instrumentation shall guarantee the following:

- Feasible installation procedure
- Durability over the monitoring period
- Protection against damage during construction
- Simple measurement handling (data acquisition and transmission)
- Accuracy as required

#### **3.5.1. Typical monitoring layouts**

In this section typical monitoring layouts for various ground and boundary conditions are shown for illustration. Typically, 3D displacement measurement (in the tunnel and if required on the surface), extensometers, inclinometers, strain gauges, piezometers, tilt meters, hose water levels, and invert probes are used to observe the system behaviour in underground structures. Additional instrumentation and tools, such as compass-clinometer or digital face mapping techniques may be required.

##### **3.5.1.1. Typical distances between monitoring sections**

Monitoring sections in tunnels and shafts are typically situated at distances of 5 - 20 m depending on the boundary conditions and requirements.

### 3.5.1.2. Examples

### Case 1: Shallow tunnel in soft ground beneath groundwater table - Figure 2

Typical hazards: surface settlements; drawdown of ground water table leading to consolidation, instability of face and unsupported ground, day lighting collapse, support failure

Focus observation on:

1. deformation of surface, buildings, and utilities
2. water table drawdown
3. face stability
4. lining displacements and integrity

Instrumentation:

Ad 1: Absolute displacement monitoring, levelling, tilt meters, hose water level, extensometers, inclinometers

Ad 2: Water level gauge, Piezometer

### Ad 3: Visual inspection

Ad 4: Absolute displacement monitoring, face displacement monitoring, invert probe, strain gauges (alternatively and/or supplementary evaluation of the displacement monitoring data), extensometer

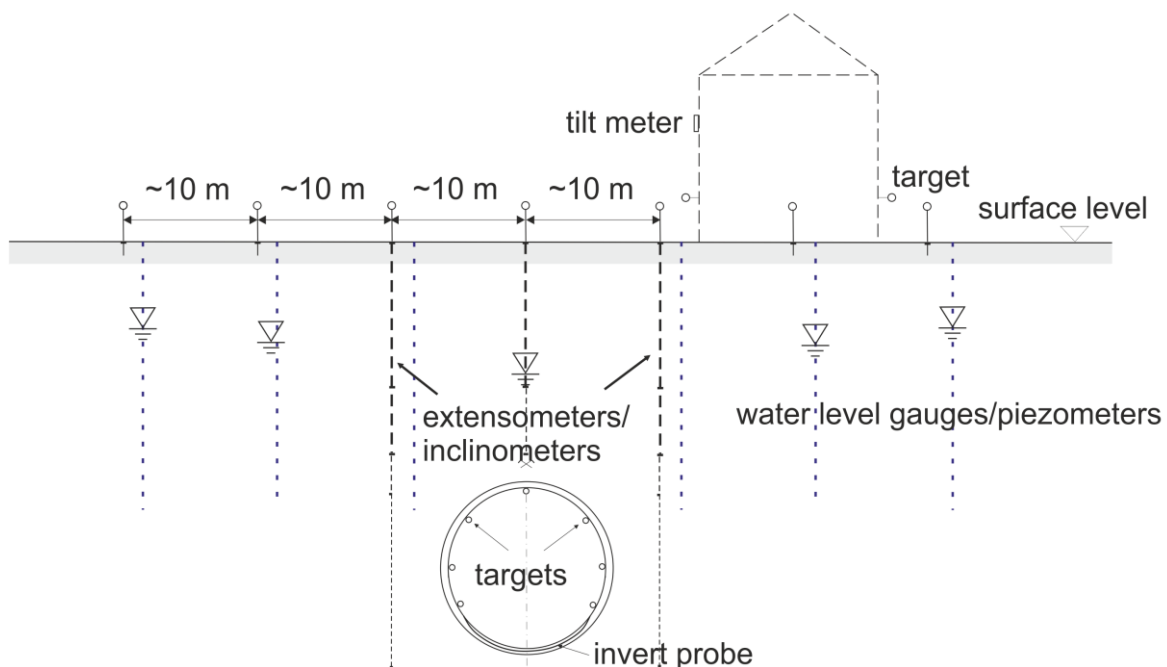


Figure 2: Surface monitoring cross section consisting of targets, extensometers, piezometers, and tilt meters on the same station of a monitoring section in the tunnel

The limitations to the layout on the surface monitoring targets are posed by the morphological conditions and land usage. Typically, the targets are organized in such manner that the con-

struction of settlement profiles (in one line or regular pattern) is possible. In addition, the displacements of buildings are measured with at least three monitoring points, in order to determine both translation and rotation.

### Case 2: Tunnel in blocky rock mass - Figure 3

Typical hazards: Discontinuity induced detachment of blocks

Focus observation on:

1. ground structure
2. discontinuity location and orientation in relation to the tunnel alignment

Instrumentation: geological compass and/or digital ground mapping

Although this type of failure in general cannot be detected in time by displacement monitoring, it is recommended to use supplemental absolute displacement monitoring, albeit in a reduced extent. This guarantees the presence of a fallback monitoring system in case of encountering adverse ground conditions.

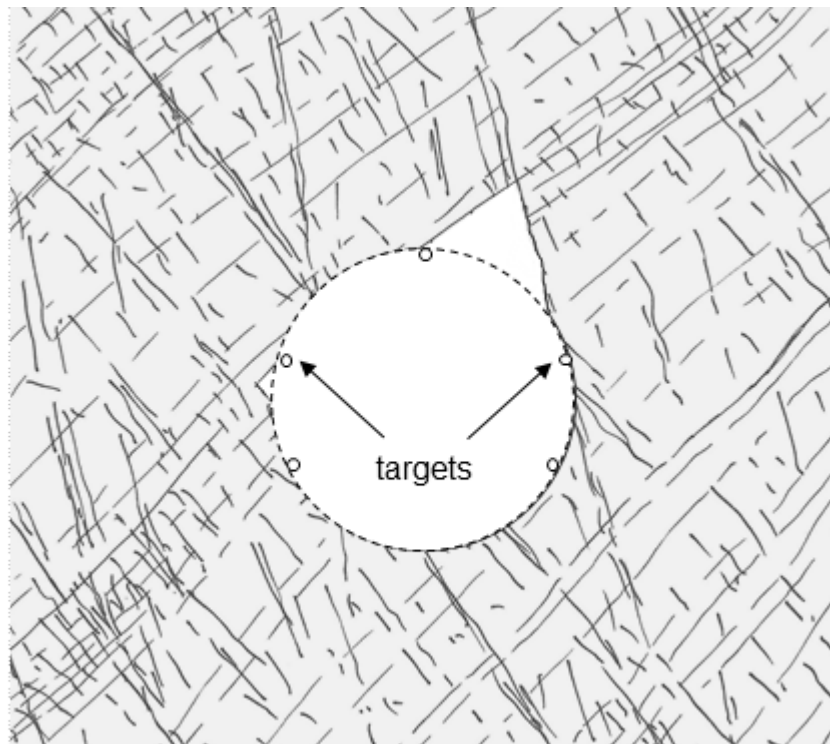


Figure 3. Blocky rock mass with potential for overbreak; with installed monitoring targets

### Case 3: Tunnel with moderate to high overburden in bedded or foliated rock mass

Typical hazards: violation of clearance profile, lining failure due to shearing along and dilation perpendicular to foliation, large anisotropic deformations

Focus of observation on:

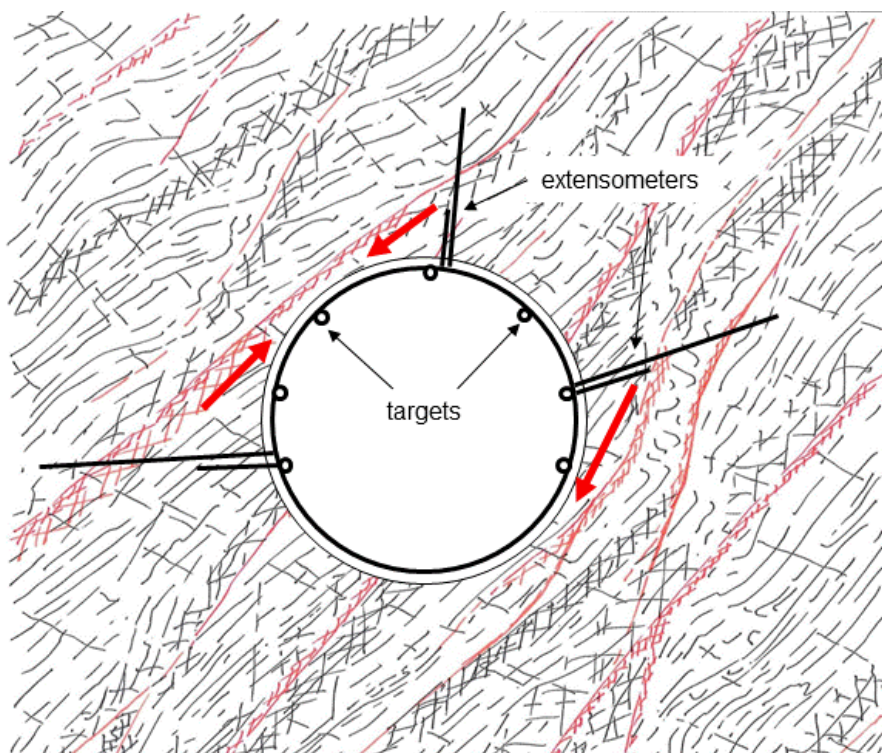
1. orientation of the bedding or foliation
2. lining displacements
3. ground displacements
4. ground structure

Instrumentation:

Ad 1 and 4: compass-clinometer or digital ground mapping

Ad 2: absolute displacement monitoring, strain gauges

Ad 3: extensometers for identification of mechanically active features



*Figure 4. Bedded/foliated rock mass with the potential to shear along faults/slickensides/foliation/bedding and dilate perpendicular to foliation/bedding; equipped with monitoring targets and optional extensometers; arrows indicate assumed shearing along faults*

#### Case 4: Tunnel in swelling ground - Figure 5

Typical hazards: clearance violation, invert failure due to long lasting increase of support loading

Focus observation on:

1. presence of water
2. mineralogical composition (swelling clays and minerals)
3. lining condition, deformation and strains
4. ground displacements

Observation/instrumentation:

Ad 1: visual inspection

Ad 2: laboratory testing

Ad 3: absolute displacement monitoring, strain meters, invert probe, fibre optical sensors

Ad 4: absolute displacement monitoring, extensometers

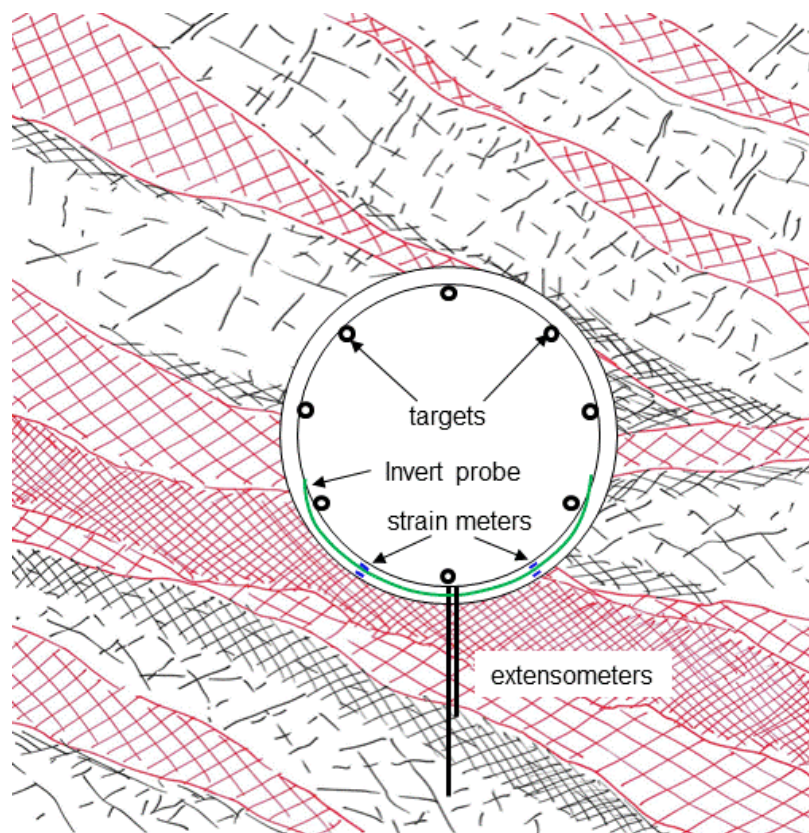


Figure 5. Rock mass with swelling potential; equipped with targets, extensometers combined with measurement point, invert probe, and strain meters installed

### 3.5.2. Reading frequency

#### 3.5.2.1. Tunnels

In general, readings close to excavation activities are taken daily; the frequency is reduced with distance to the face and decreasing rates of displacements. Shorter monitoring intervals can be required due to project specific requirements.

A possible concept showing minimum reading frequencies and monitoring ranges for surface and underground monitoring for a top heading-bench-invert sequence is illustrated in Figure 6 and summarized in Table 3 and Table 4.

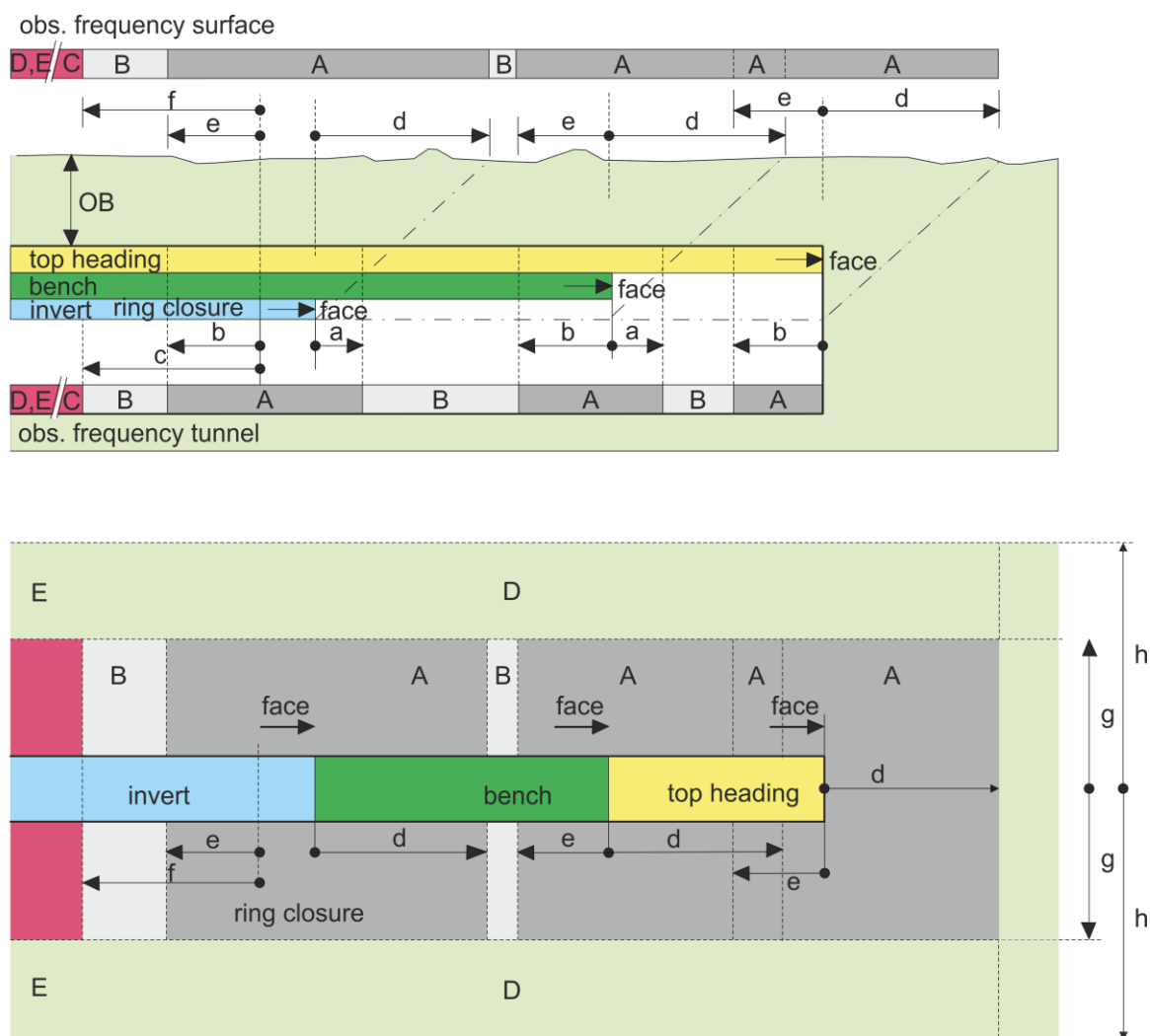


Figure 6. Sketch of areas with different reading frequencies for a shallow tunnel; longitudinal section (top), and plan view on surface level (bottom); OB: overburden

It is distinguished between zones, directly influenced by tunnel excavation, and zones which are outside of this range but within a fixed observation corridor (i.e. monitoring points on buildings). The values are minimum requirements, which are increased by the parameters “Xi”

(variable lengths) and “n” (number of monitoring sections (MS)) to consider influences of lowering of the ground water, building location, special ground conditions, etc. The parameters “Xi” have to be determined on a project specific basis. For other construction sequences, like side wall galleries, parallel headings, etc. the above recommendations also apply.

*Table 3. Spatial differentiation of monitoring ranges related to construction phases*

	distances	monitoring range
Tunnel (dia: tunnel diameter)	a	$2dia+X_1$ and min. (2+n)MS
	b	$3dia+X_2$ and min. (3+n)MS
	c	$5dia+X_3$ and min. (5+n)MS
surface	d	$dia+OB+X_4$
	e	$3dia+X_5$
	f	$5dia+X_6$
	g	$1,5dia+OB+X_7$
	h	$3dia+2OB+X_8$

*Table 4. Typical minimum reading frequencies in the respective monitoring ranges from the geotechnical point of view*

range	min. observation frequency
A	1 per day
B	2 per week
C	1 per week
D	1 per month
E	as required

#### **3.5.2.2. Shafts**

The measuring frequency is the same as for a top heading advance. Readings of the monitoring points on the surface should be taken daily. With decreasing influence of the excavation on the surface movements, the reading frequency can be reduced.

#### **3.5.2.3. Buildings, utilities, traffic infrastructure**

In general the reading frequency shall be according to chapter 3.5.2.1, respectively according to intervals prescribed by the authorities. Sensitive buildings may require a higher reading frequency.

#### **3.5.2.4. Measurements during additional mining activities**

When excavating niches, cross passages or galleries intersecting a tunnel or shaft, the monitoring devices in the influenced zone should be monitored on a daily basis, starting prior to the beginning of the work until stabilization has been obtained in the area of observation.

#### **3.5.2.5. Long term monitoring**

In ground with pronounced time dependent behaviour (swelling ground, ground with tendency for creeping, consolidation, etc.) the system should be monitored also after completion of construction works. It may be required to continue monitoring during operation.

## 4. MONITORING METHODS

### 4.1. Absolute 3D Displacement Monitoring

Over the last two decades 3D displacement monitoring has become common practice, and due to the high information content, has gradually replaced other techniques.

#### 4.1.1.1. Basic principles

The measurements are executed by using a total station (tachymeter) and targets [4]. Precise prism-targets as well as bireflex-targets (reflectors) are used and their spatial position within the global or project coordinate system determined. Discrete three dimensional displacement measurements are executed by repeated measurements (generally on a daily basis). As the complete monitoring generally cannot be carried out from one position, an interlinked observation scheme is required, which is established using identical reference points (Figure 7). Stable reference points are differentiated from points which are still moving. Points with a defined maximum displacement rate (generally  $< 1\text{mm/month}$ ) can be used as reference points.

#### 4.1.2. Requirements

The principle of the “over determined free stationing” is used to obtain the instrument position.

The absolute position of all coordinate components of marked measuring points shall be determined with an accuracy of  $\pm 1\text{mm}$  (standard deviation) in relation to neighbouring measuring sections over the whole observation period.

To obtain this accuracy, the following conditions should be observed:

- Distance between instrument and closest reference point: 10 - 30 m
- Minimum distance to farthest reference point: 90 m
- Maximum distance to monitoring points: 80 m
- Maximum distance between instrument positions: 110 m
- Use approximately same instrument positions for consecutive readings
- Position instrument on stable ground
- Start free stationing from closest and finish at farthest targets
- Displacement measurement starting from the farthest target
- Make connection sightings approximately symmetrical to tunnel axis
- Intermediate orientation measurements and closing check to selected reference points
- Make zero readings immediately after installation of targets (latest prior to next excavation)
- Measuring of all points in one section (in case of zero readings in the bench, top heading targets shall be measured as well)
- Recording of meteorological conditions for consideration in the evaluation
- Acclimatization of instrument to avoid deviations due to temperature change

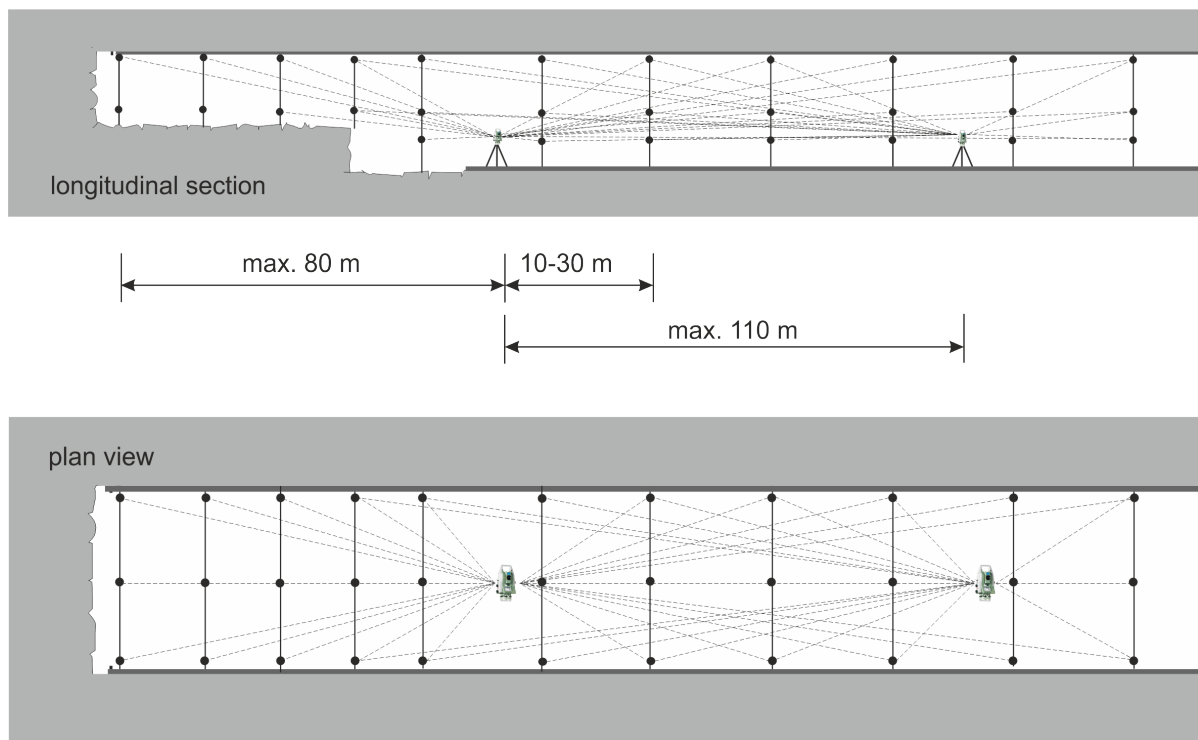


Figure 7. Sketch of an interlinked free station method in the tunnel

The following sources for errors should be avoided:

- Sightings close to the tunnel wall (minimum distance sighting to wall 0.5 m to 1 m)
- Measurement errors due to refraction (i.e. sighting through or close to heat sources)
- Instrument position close to sidewall
- Asymmetrical connection sightings (see Figure 8)
- Measurements in very dusty environment or during strong vibrations (i.e. caused by machinery)

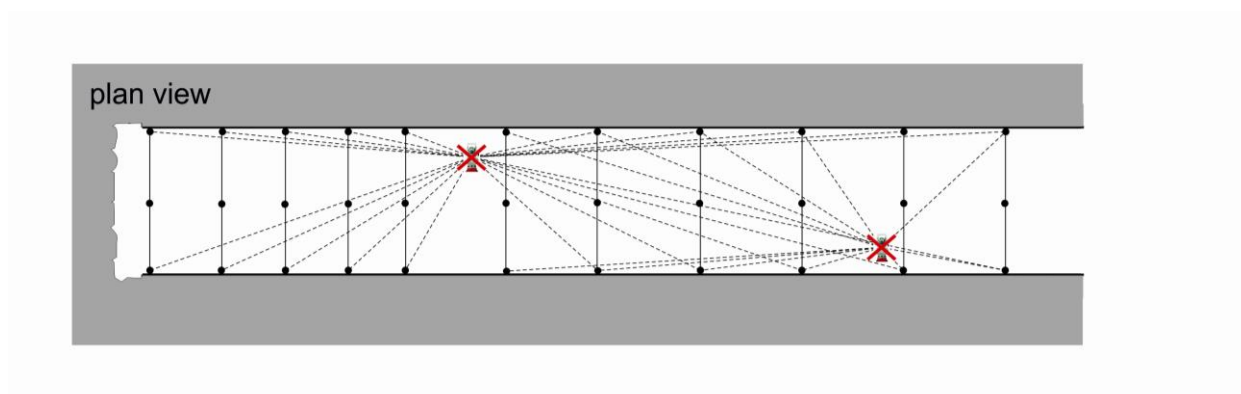


Figure 8. Sketch of asymmetrical connection sightings (to be avoided)

The surveyor shall record and submit the following items after each measuring campaign:

- System of measuring sequence (related to measuring section or along the tunnel)
- Points not measured and indication of reason (destroyed, not visible, etc.)
- Significant displacements (measuring error, rapid increase in displacements)
- Zero readings
- Monitoring conditions (air quality, vibrations, limited visibility, heat sources, etc.)

#### 4.1.3. Tachymeter

An electronic total station (Figure 9) with automatic recording unit should be used having following minimum precision:

Horizontal angle measurement:  $\pm 1''$  (0.3 mgon)

Vertical angle measurement:  $\pm 1''$  (0.3 mgon)

Distance measurement:  $\pm (1 \text{ mm} + 1.5 \text{ ppm})$



Figure 9. Tachymeter, example Leica, TS 09

#### 4.1.4. Targets

The targets are mounted on special bolts with an adapter (see Figure 10).

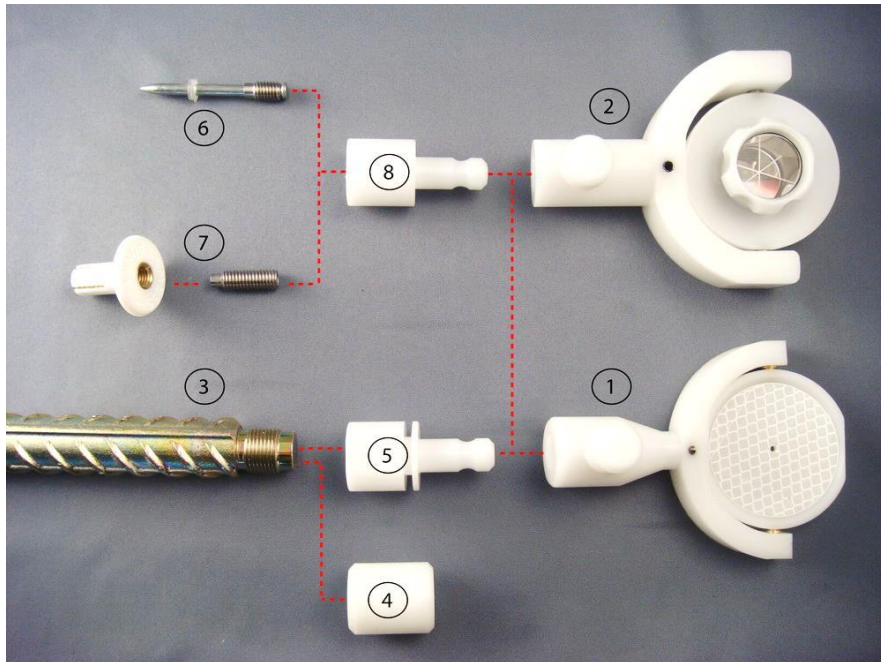


Figure 10. Components of 3D displacement monitoring targets; bi-reflex (1) and prism target (2), bolt (3) with protecting cap (4) and predetermined breaking point (5), mounting bolts (6 + 7) with adapter (8)

The minimum manufacturing precisions required are:

- Adapter with breaking point:  $\pm 0.1$  mm
- Triple prism target:  $\pm 0.01$  mm
- Bireflex target:  $\pm 0.1$  mm

Required minimum repeatability of readings:

- Triple prism target:  $\pm 0.1$  mm
- Bireflex target:  $\pm 0.3$  mm

This precision shall apply for repeated readings when targets are changed in their orientation or targets/ adapters are replaced.

#### 4.1.5. Installation and measurement procedure

To achieve maximum output from the measured data and to ensure comparable measurement results, the following should be observed during underground and surface monitoring:

- A predefined breaking point has to be provided between the bolt and the target, in order to prevent bolt damage
- Solid embedment of the bolt

- Protection against damage during shotcrete installation
- Installed utilities have to be considered when positioning the targets, to guarantee sightings by the tachymeter
- In case of surface targets, the foundations have to be embedded below the frost or permafrost zone
- For surface targets, the target height above ground should be between 50 and 100 cm (influence of refraction)

#### 4.1.5.1. Underground

- The targets of a monitoring section should be
  - Installed immediately behind the face of the last round and the zero reading taken without delay; deviations from the planned station installation of maximum  $\pm 1$  m are acceptable
  - Installed at the same position as those of previous construction phases, i.e. top-heading, bench (tolerance  $\pm 1$  m)
  - Installed at the same station as the surface monitoring points (tolerance  $\pm 1$  m)
- Consistent recording of construction phase (e.g. face positions) and exact assignment to measurements
  - The position of the face is determined by the mean value of at least three face position measurements (Figure 11).
  - Face position can be determined without target (precision  $\pm 10$  cm)
  - The new face position is valid as soon as more than 25% of the face is excavated

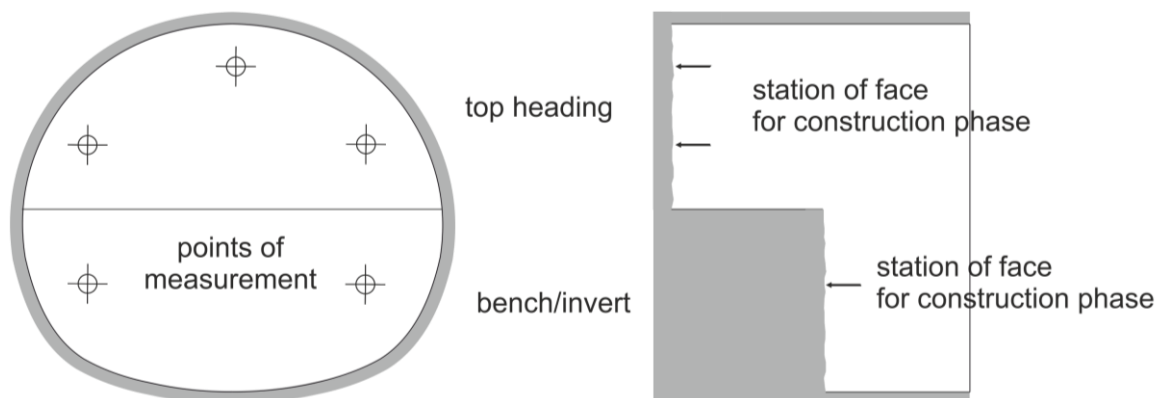


Figure 11. Determination of the face position

Figure 12 and Figure 13 show the common practice of installation and protection of a monitoring bolt against damage in shotcrete linings. The monitoring bolt and protection cap is fixed to the wire mesh. During shotcreting the bolt should be covered appropriately. The bolt head should be recessed at a minimum 10 mm from the inner surface of the shotcrete lining.

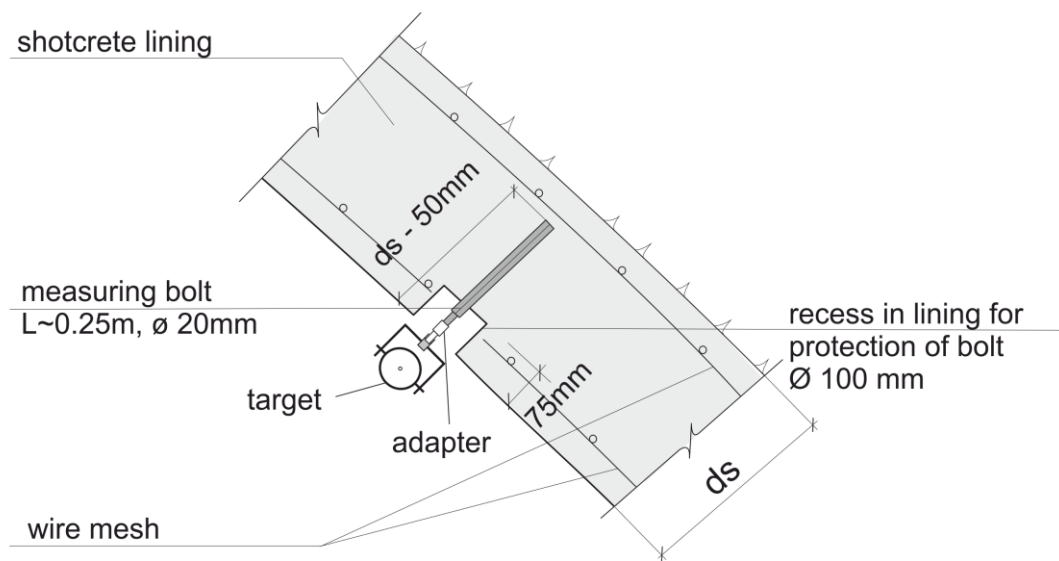


Figure 12. Sketch of monitoring bolt unit mounted in shotcrete lining



Figure 13. Bolt and adapter with prism target mounted in recess

#### 4.1.5.2. Arrangement of monitoring displacement targets

Figures 14 – 17 depict different examples for the arrangement of 3D displacement monitoring targets in tunnels of different size, caverns and shafts. The sketches give an indication of the minimum requirements, which can be adapted in accordance to the local conditions in terms of underground conditions and sensitivity of adjacent infrastructures.

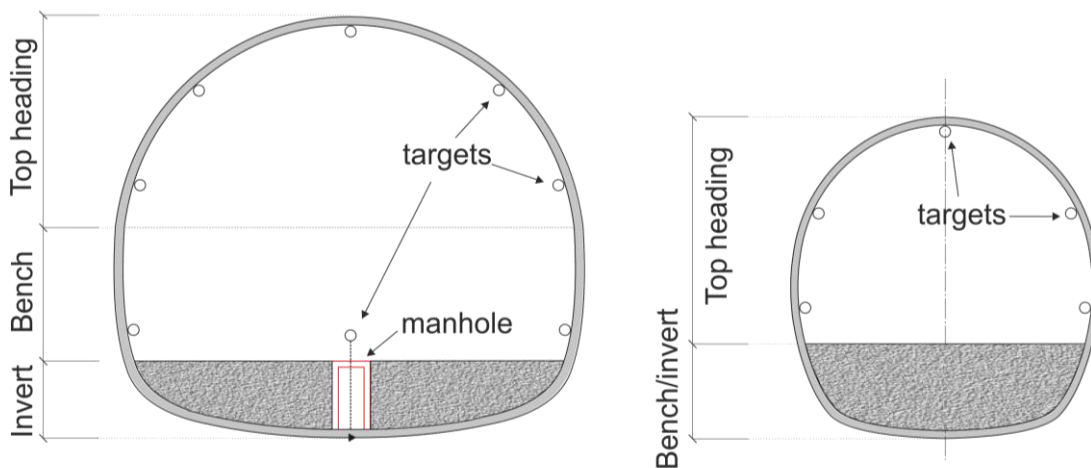


Figure 14. Sketch of monitoring layout for tunnels or galleries with a top heading, bench and invert excavation sequence

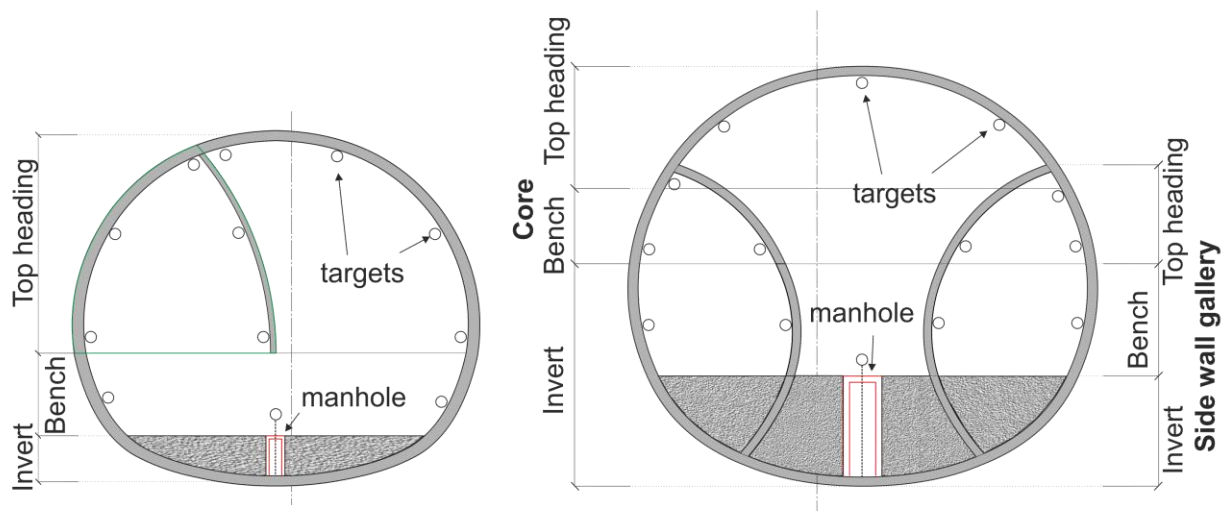


Figure 15. Sketch of monitoring layout for cavern or tunnel with sidewall galleries



*Figure 16. Arrangement of 3D displacement monitoring targets in a tunnel with side wall galleries during construction as designed in Figure 15 (right)*

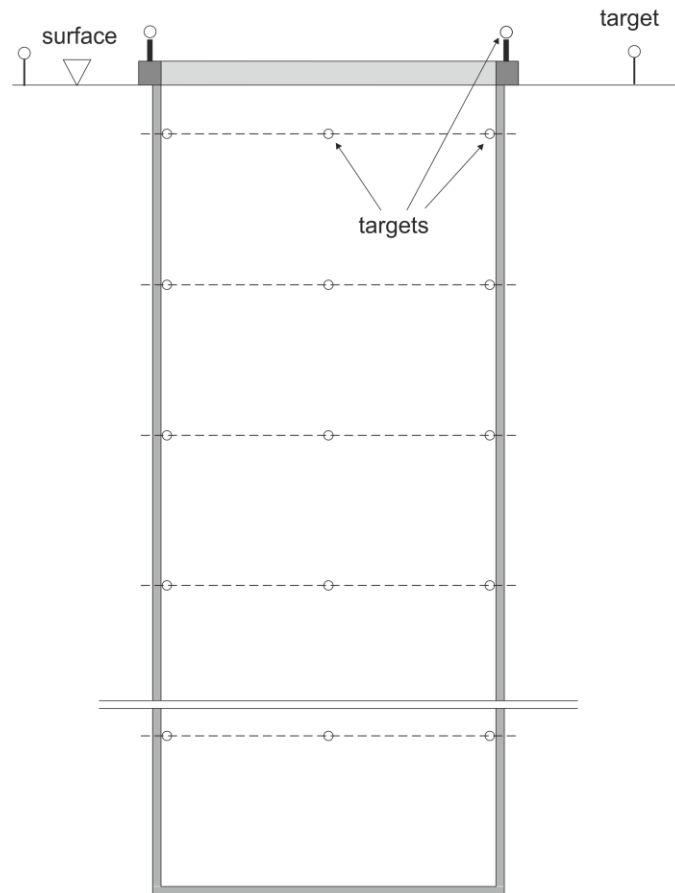


Figure 17. Sketch of monitoring layout for a circular shaft

#### 4.1.5.3. Surface

Installation of the instruments and the zero readings of surface measurements shall be done well ahead of the

- Excavation and / or
- Ground water table lowering

It is advisable to take at least two readings, which are not influenced by construction activities. Additional measurements may be required at request of the authorities.

##### 4.1.5.3.1. Surface target installation

The following figures depict various types of possible monitoring target installation, suitable for application on native ground surfaces, traffic ways, objects and railways.

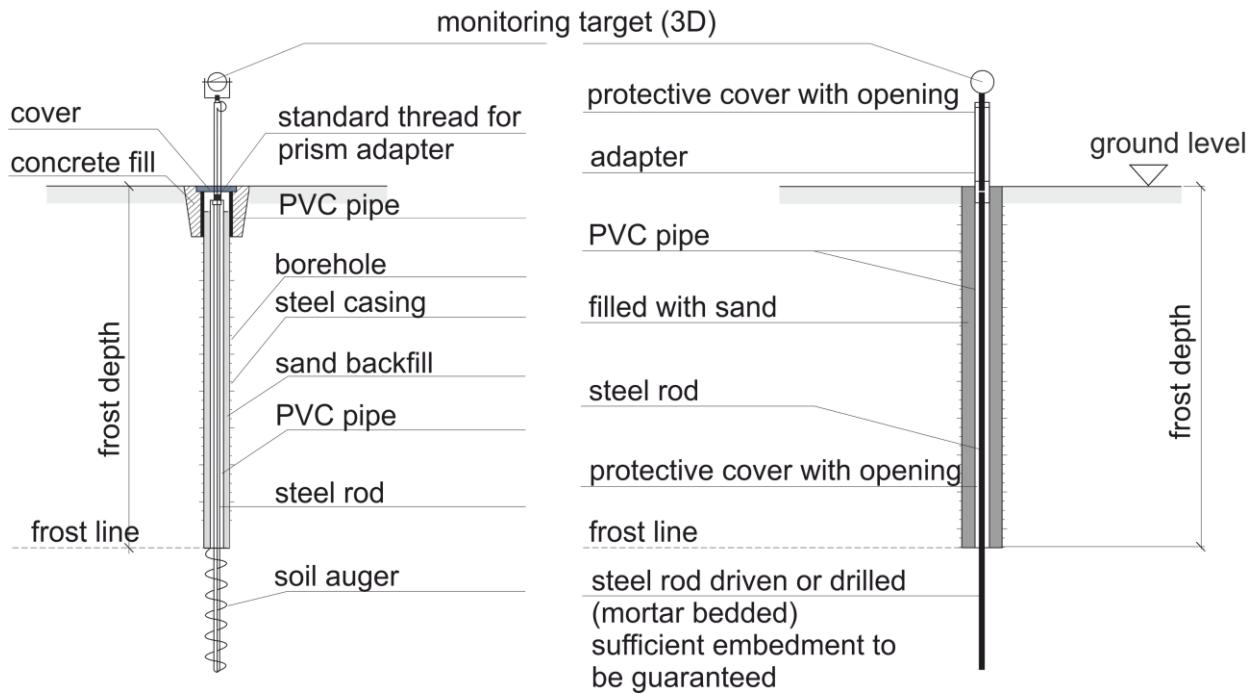


Figure 18. Sketch of surface settlement points (native ground)

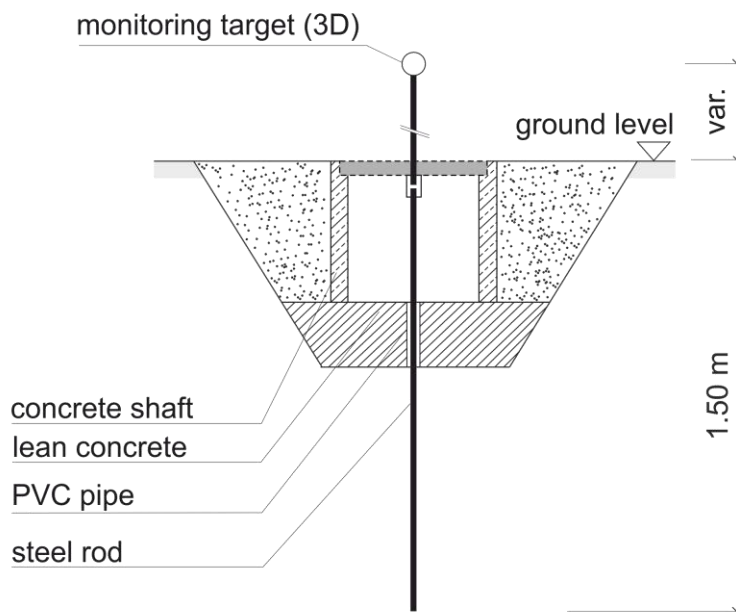


Figure 19. Sketch of a surface monitoring target in public areas

#### 4.1.6. Online-Geomonitoring

Online geomonitoring systems feature an automated measurement data acquisition and analysis system (Figure 20, Figure 21). It requires the same equipment as described above, and can be used both for underground and surface monitoring. Automatic information can be provided to the stakeholders if certain pre-defined trigger values are exceeded.



Figure 20. Example for an online monitoring system [5] for the continuous observation of surface displacements in a critical area of a tunnel project with shallow overburden

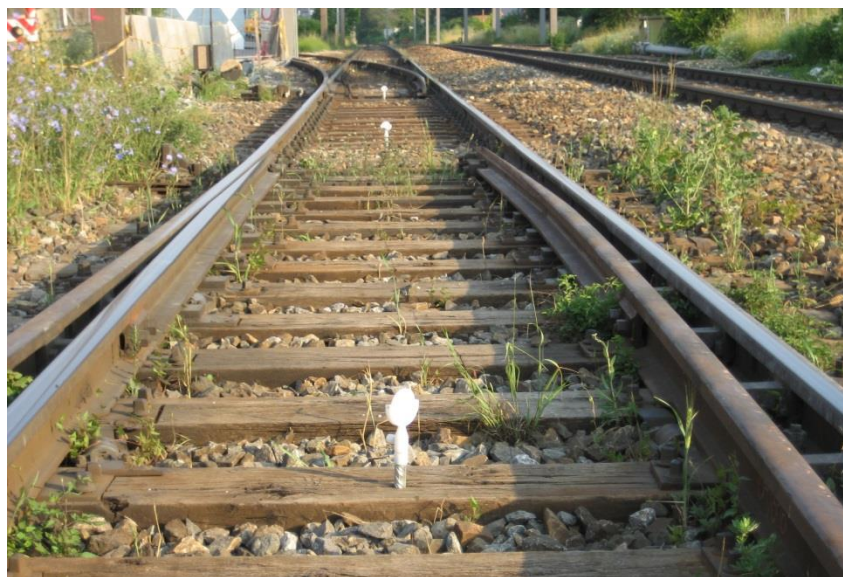


Figure 21. Targets mounted on sleepers

## 4.2. Face displacement monitoring

The face monitoring system requires an automated measurement data acquisition system which is conducting continuous tachymeter measurements without targets [6, 7]. The measurements taken on temporary obstacles should be automatically filtered (Figure 22). The maximum range is limited by the reflector-less distance measurement (approx. 100 m) and the accuracy of the results is  $\pm 1\text{--}3\text{ mm}$ , depending on the distance and on atmospheric conditions in the tunnel.

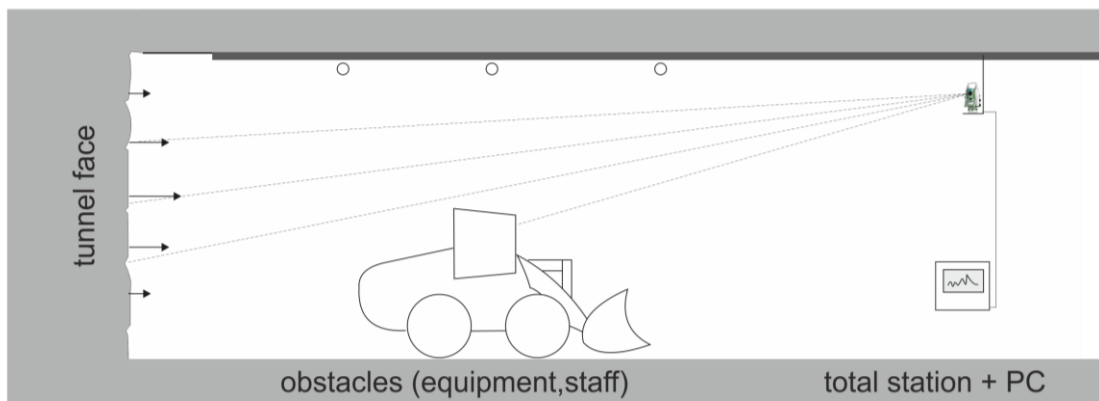


Figure 22. Sketch of the face monitoring system, modified after [8]

## 4.3. Levelling

For precise levelling, an electronic level with automatic recording unit in combination with an invar staff and marked points is used. The installation procedures presented in section 4.1.5.3.1 can be used for levelling as well. Required accuracy for a double levelling procedure (standard deviation) per 1 km:  $\pm 1\text{ mm}$

## 4.4. Extensometers

Extensometers are used for determining ground movements by measuring the shortening or elongation (relative displacement) between two points. They allow an assessment of the development of strains and stabilisation of movements in the surrounding ground.

### 4.4.1. Types of extensometers

There are various types of extensometers; however all of them yield the same measurement data. Typical extensometer types are:

- Single or multi point rod extensometers, consisting of mechanically anchored measurement rod(s) and a protective pipe, allowing unhindered relative movement (Figure 23)

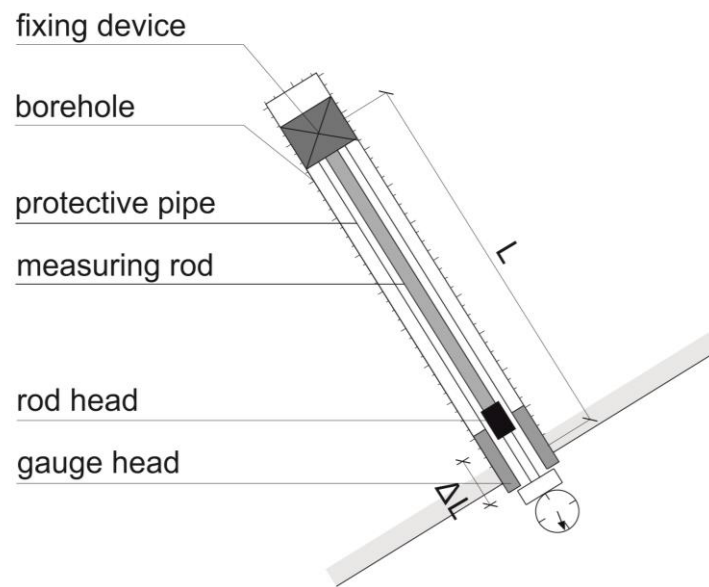


Figure 23. Schematic layout and principle of a single point rod extensometer

- Probe extensometers, consisting of a probe guide tube, points connected to the ground and a moveable reading element  
The measurement principles vary (mechanical, non-mechanical, etc.). Typical examples of such instruments are sliding micrometre probes and magnetostrictive probes. They allow measuring at several depths along one probe [9].

#### 4.4.2. Installation requirements

The following installation requirements should be observed:

- In case of rod extensometers, the head of the instrument shall be protected by a cap. A reliable bond between the anchors and the ground shall be ensured
- The extensometer head should be paired with a 3D displacement monitoring target. In case of surface installation, levelling of the extensometer head shall be conducted concurrently
- The position of the extensometer head should be determined immediately after installation, and parallel to the zero measurement of the extensometer
- The spatial orientation of the extensometer shall be measured

#### 4.4.3. Accuracy

Common resolution of reading instrument: 0.01 mm

## 4.5. Anchor load cells

Anchor load cells measure the loads in anchors and are installed at the anchor head (Figure 24).

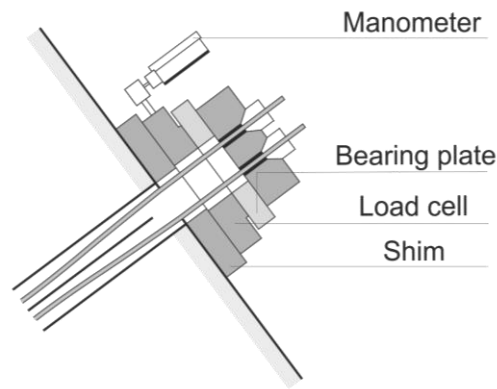


Figure 24. Schematic view of a load cell for a prestressed anchor

### 4.5.1. Types of load cells

Measurement of the loads is either done by a cell filled with oil, or cells equipped with strain gauges. Values can be either read out by manometer or remotely.

### 4.5.2. Installation requirements

The surfaces shall be planar, and the read out device accessible.

### 4.5.3. Accuracy

Accuracy of hydraulic systems is  $\pm 1\%$  of the capacity of the load cell. Accuracy for electrical load cells is in the range of  $\pm 0.5\%$ . Error due to temperature change of  $20^\circ$  for both systems is in the range of  $1.2\%$ .

## 4.6. Tilt meters

Tilt meters are used for measuring very small changes of the inclination, either on the ground or in structures. Such changes may be caused by excavation, tunnelling, dewatering, or loading of the structure.

### 4.6.1. Installation requirements

The following installation requirements should be observed:

- The position of the tilt meter should be determined immediately after installation and parallel to the zero measurement of the tilt meter

#### 4.6.2. Types of tilt meters

There are different types of tilt meters:

- Electro level tilt meter
- Optoelectronic tilt meters

#### 4.6.3. Accuracy

The total system accuracy of tilt meters is between 0.005 mrad and 0.050 mrad, depending on the measuring range.

### 4.7. Hydrostatic levelling system

Water levels are used for the continuous monitoring of changes in height of buildings and other technical constructions (Figure 25).

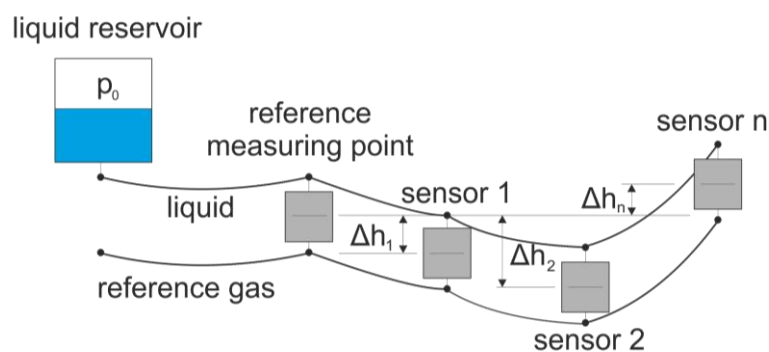


Figure 25. Schematic layout and principle of a hydrostatic levelling system [10]

#### 4.7.1. Installation requirements

The following installation requirements should be observed:

- For temperatures under 1°C a special non freezable liquid shall be used
- The sensor positions should be paired with a 3D displacement monitoring target
- The zero reading of the monitoring targets should be done immediately after installation and parallel to the zero measurement of the water level

#### 4.7.2. Types of hose water levels

The following types of water levels are used:

- Hydrostatic levelling system using manometers (Figure 25)
- Electrical water level systems
- Mechanical water levels

### 4.7.3. Accuracy requirements

The total system accuracy for hydrostatic water levelling systems is 0.3 mm for the measured height differences. The accuracy of hose water levels varies between 1 and 2 mm. The measuring range for all systems is about 200 mm.

## 4.8. Borehole Inclinometers

Inclinometers are used for determining ground movements by measuring angular deflections (Figure 26). They are commonly installed from the surface in the vertical and horizontal direction. Also the installation from the tunnel in +/- horizontal direction is possible. Combination with extensometers (sliding micrometre) is possible.

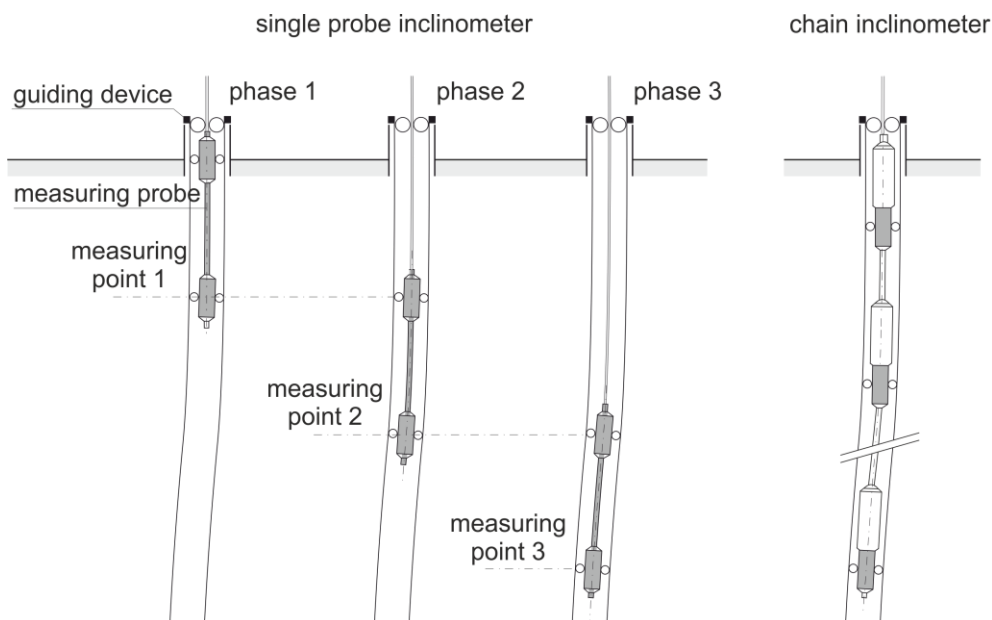


Figure 26. Principle of single probe and chain borehole inclinometers

### 4.8.1. Types of inclinometers

There are two main types of inclinometers:

- Single probe inclinometers
- In-place inclinometers (chain inclinometer)

### 4.8.2. Installation requirements

The following installation requirements should be observed:

- The inclinometer head should be paired with a 3D displacement monitoring target (Figure 27)
- The position of the inclinometer head should be determined immediately after the installation and parallel to the zero measurement of the inclinometer

- The casing slots should be oriented such that they are parallel and perpendicular to the orientation of any tunnel excavation or wall to be monitored



Figure 27. Head of borehole inclinometer with 3D displacement monitoring target (Photo: Weissnar)

#### 4.8.3. Accuracy requirements

- Common resolution of reading instrument:  $\pm 0,02$  mm per 500 mm gauge length
- The system accuracy shall be better than 1 mm per 10 m measuring length
- Measuring range: vertical or horizontal:  $\pm 30^\circ$  ( $10^\circ$  for in-place inclinometers)

#### 4.9. Water level gauges and Piezometers

Water level gauges are used to measure the water level in a borehole, by using a light plummet or sounding device. Piezometers are used for determining ground water pressure within a borehole (Figure 28).

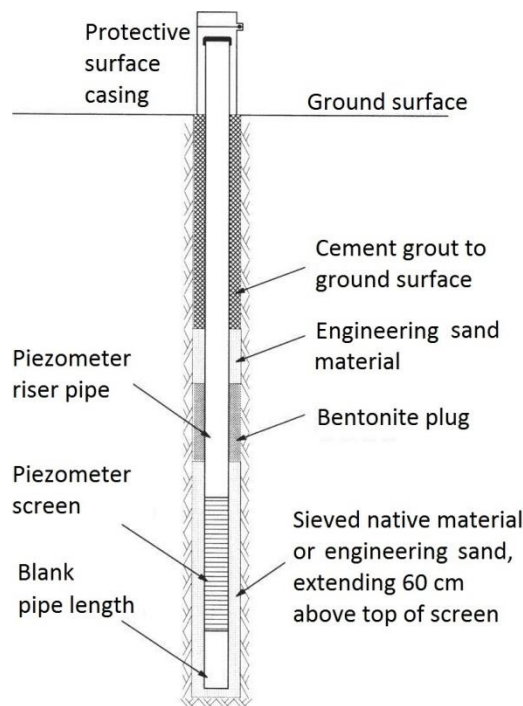


Figure 28. Schematic layout and principle of a standpipe piezometer

#### 4.9.1. Types of piezometers

The following types are commonly used:

- Single or multiple standpipe piezometer (Casagrande type)
- Standpipe
- Vibrating wire

#### 4.9.2. Installation requirements

The following installation requirements should be observed:

- The position of the piezometer head should be determined immediately after installation
- The depth of the probe within the borehole should be recorded

#### 4.9.3. Accuracy requirements

The system specifications should meet following requirements:

- Common resolution of reading instrument:  $\pm 0.025$  % of full scale
- The system accuracy shall be better than  $\pm 1$  % of the full scale
- The operating range of the system shall not exceed the expected maximum pressure by more than 50 %

## 4.10. Strain gauges

The following strain gauges are commonly used:

- Strain meters
- Vibrating wires (for cured concrete structures e.g. inner lining)
- Fibre optical sensors

### 4.10.1. Strain meters

Strain meters are used for measuring strains in shotcrete or inner linings (see Figure 29).

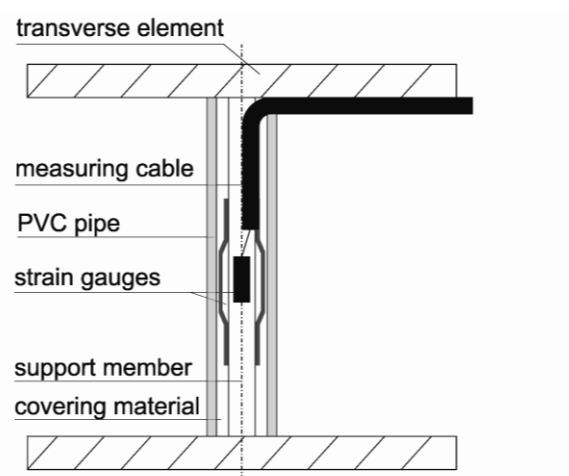


Figure 29. Schematic layout of a shotcrete strain meter (figure courtesy of Geodata)

#### 4.10.1.1. Accuracy requirements

Shotcrete strain gauges should have a minimum measuring range of 1.0 % in compression and 0.2 % in tension. They should be temperature compensated to account for temperature increase of the shotcrete during the hydration process.

- Resolution of reading instrument:  $\pm 0.01$  % of full scale
- The system accuracy shall be better than  $\pm 1$  % of full scale

### 4.10.2. Vibrating wire sensors

Vibrating wire sensors are used to measure length changes along a measuring line. The layout is the same as for strain meters.

#### 4.10.2.1. Accuracy requirements

The system specifications should meet the following requirements:

- Resolution of reading instrument:  $\pm 0.01$  % of full scale
- The system accuracy shall be better than  $\pm 1$  % of full scale

#### 4.10.3. Fibre optic sensors

Fibre optical sensors are used to measure length changes along a measuring line [11, 12].

##### 4.10.3.1. Accuracy requirements

The required accuracy strongly depends on the application and thus different levels of accuracy are needed. As fibre optic sensors are temperature sensitive, temperature measurements always have to accompany strain measurements with fibre optical sensors. Alternatively, a self-compensating configuration can be chosen.

For strain measurements in structural elements, like concrete, steel or shotcrete, the following requirements should be met:

- System accuracy: 1  $\mu\text{m}/\text{m}$

For measurements of ground movements, the following requirements should be met:

- System accuracy: better than 0.1 mm/m



Figure 30. Fibre optic sensor installed

#### 4.10.4. Installation requirements

The following installation requirements should be observed:

- The position of the sensors should be determined and recorded
- The sensors should be fully embedded in the lining/ground
- Wires should be protected against damage



Figure 31. Pair wise installed strain gauges in the shotcrete lining of a shaft and arrangement of the wiring

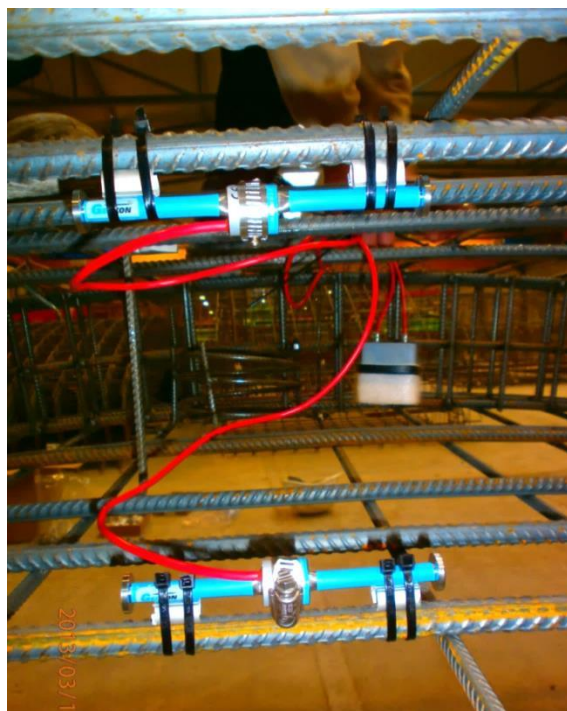


Figure 32. Vibrating wire sensors attached to reinforcement

#### 4.11. Invert probe

The invert probe is used for verifying the integrity of the invert (Figure 41), which usually is covered with roadway material and therefore cannot be inspected visually. The device consists of a cylindrical probe which is connected to a stranded wire and inserted in a PVC-hose.

#### 4.11.1. Installation requirements

The following installation requirements should be observed:

- The device shall be installed together with the shotcrete lining
- Both ends of the hose shall be accessible, protected against damage and covered with a cap

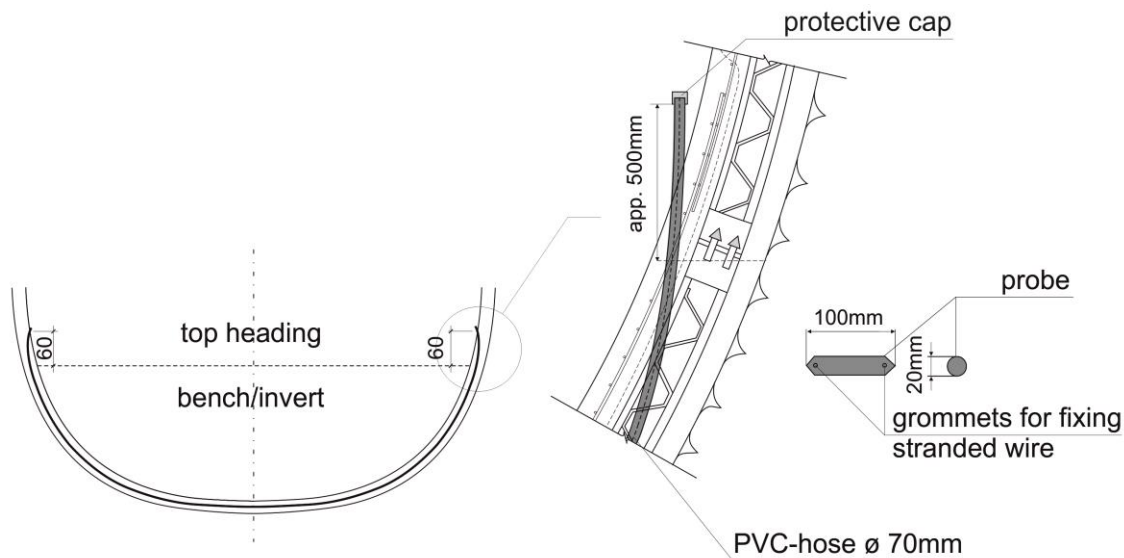


Figure 33. Typical invert probe (details and arrangement in cross section)

#### 4.12. Measurements of discontinuity location and orientation

An essential task to evaluate discontinuity controlled failure modes is the determination of the spatial orientation and location of discontinuities. Conventional rock mass characterization requires physical access to the rock surface for applying a compass-clinometer device.

##### 4.12.1. Manual structure mapping

The acquisition of spatial orientation of discontinuities is done with the help of a compass-clinometer device (Figure 34). The device is located on a discontinuity (Figure 35), and dip and dip direction read from the instrument. The result usually is plotted in a face sketch.



Figure 34. Compass-clinometer (Clar system)

Accuracy of measurement: reading accuracy is approximately  $1^\circ$  for the dip direction, and approximately  $2^\circ$ - $3^\circ$  for the dip angle. Deviations in the orientation measurement can be caused by magnetic field or steel elements installed in the tunnel.

Shortcomings of this measuring technique are:

- Readings taken manually in potentially hazardous area
- Readings only possible in accessible areas
- Readings not reproducible after excavation of next round
- Location of features generally only estimated



Figure 35. Measuring of discontinuity orientation with compass-clinometer device at a tunnel face (Photo: Geoteam Koralmtunnel, lot 3)

#### 4.12.2. Digital ground mapping

Two technologies are mainly used for contact free (remote) measuring of discontinuities and other geological features: Digital photogrammetry [13, 14] and LiDAR (light detection and ranging).

Major advantages of these remote techniques include:

- The ability to quickly acquire large portions of rock masses including inaccessible areas
- The possibility to zoom in and out of an outcrop, which leads to a better understanding of large features
- Permanent documentation of the rock face condition and excavation stages for reporting and contractual-legal issues

The general work-flow when applying these techniques includes collecting data by taking photographs or scanning the region of interest from distant locations and processing the acquired data to a 3D surface description (3D image) with a photo overlay (Figure 36).

##### 4.12.2.1. Accuracy

In the context of remotely acquiring surfaces for geologic mapping, accuracy shall be seen two-fold: (a) the accuracy of single surface point measurements (positional accuracy), and (b) the accuracy of the entire surface description (shape accuracy) which is strongly related to the term resolution or point density.

Both mentioned technologies have the capability to scan/acquire surfaces at different resolutions and different positional accuracies. The higher the resolution, the smaller are the details that can be mapped from the resulting 3D data. It is therefore reasonable to consider the requirements of a project first and then choose the measurement/documentation tool and its resolution accordingly.

In the case of tunnel face mapping, it is generally sufficient to map geologic structures in the sub-centimetre range. Assuming the sectional area of a tunnel being about  $60 \text{ m}^2$  and it is photographed with an 18 MPix camera, a resolution of 3-5 mm/pixel and a positional accuracy error in the same range can be expected. With LiDAR similar resolution can be achieved.



*Figure 36. 3D image of a tunnel face and lining (Photo: 3GSM)*

## 5. EVALUATION, PRESENTATION AND INTERPRETATION OF MONITORING DATA

This chapter presents the state-of-the art methods of evaluation, presentation and interpretation of monitoring data.

Additional processing and interpretation of the monitoring results assist in

- Understanding mechanical processes
- Identifying “normal” behaviour
- Timely detection of deviations
- Assessing and predicting of displacement development and ground conditions surrounding the tunnel

Continuous analysis of results and their interpretation can be used to verify the design assumptions and predicted system behaviour and ensure that the structural integrity fulfils safety, serviceability and economical requirements. The methods required for achieving the goals listed above are described and discussed in this chapter. Selected case histories illustrate the process.

### 5.1. General displacement development

Figure 37 depicts a typical radial displacement development. The  $Z_0$ -line represents the total displacements caused by the excavation of a tunnel. The total displacements consist of the displacement occurring ahead of the face ( $Z_{P0}$ ), the displacement occurring between excavation and zero reading ( $Z_{P1}$ ), and the monitored displacements after zero reading ( $Z_m = Z_1$ ).

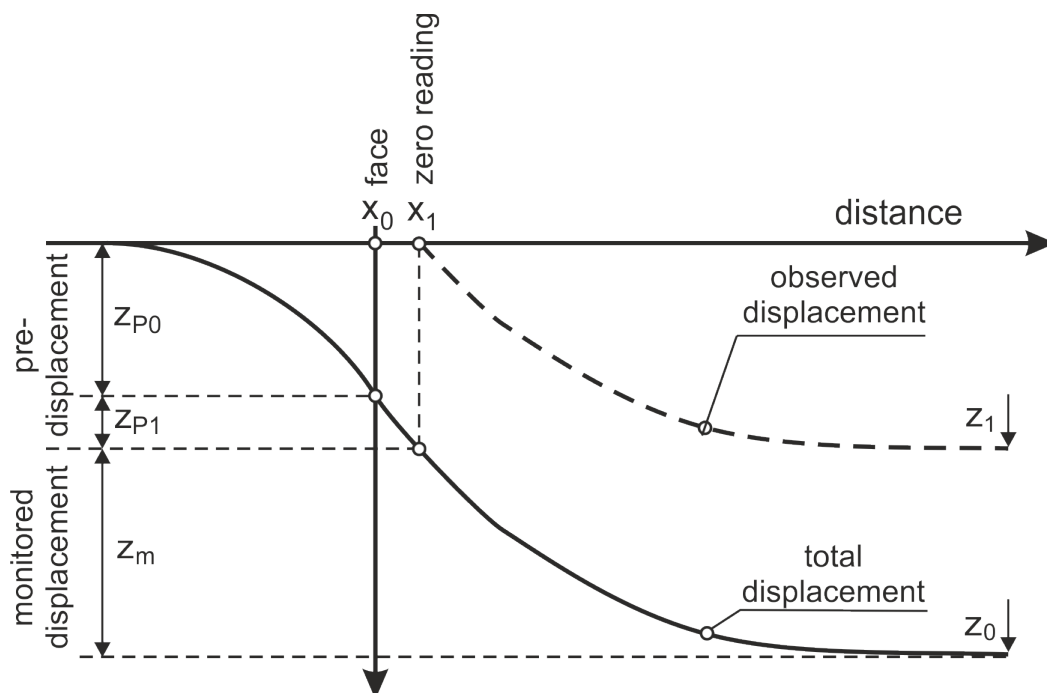


Figure 37. Typical radial displacement development

## **5.2. 3D displacement monitoring**

Various methods of visualisation can be used to verify the stabilization process, and their benefits and shortcomings are shown and discussed.

### **5.2.1. Time - displacement diagram**

Time - displacement diagrams are used to present vertical, horizontal and longitudinal displacement components versus time. Typically, the results of the displacement measurements of one or all targets in one monitoring cross section are plotted in a single diagram. Construction phases are also presented on the same plot allowing correlation between construction activities and displacements. The displacement history is used to assess the stabilisation process.

### **5.2.2. Distance - displacement diagram**

Distance – displacement diagrams are used to assess the stabilization process in relation to the face advance. More than one component or target can be shown on the plot of one section. In addition, displacements of different sections can be shown on a single plot.

### **5.2.3. Assessment of stabilization**

As a general rule, displacement increments decrease with the increasing distance between the face and the observed section.

#### **5.2.3.1. *Continuous advance***

A typical displacement development versus time and face distance show Figure 38 and Figure 39.

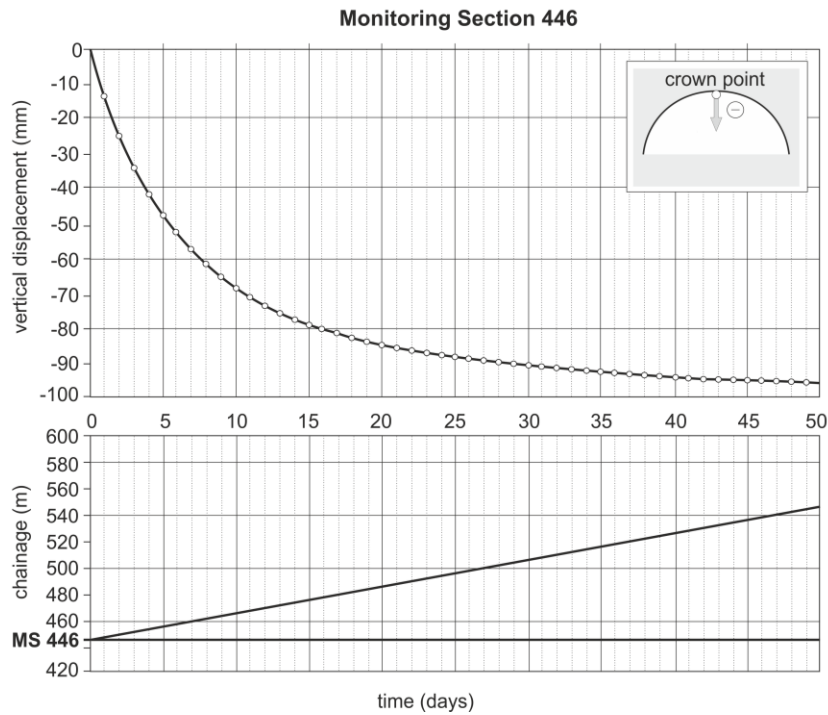


Figure 38. Typical time-displacement diagram: stabilisation of the top heading advance with continuous face advance

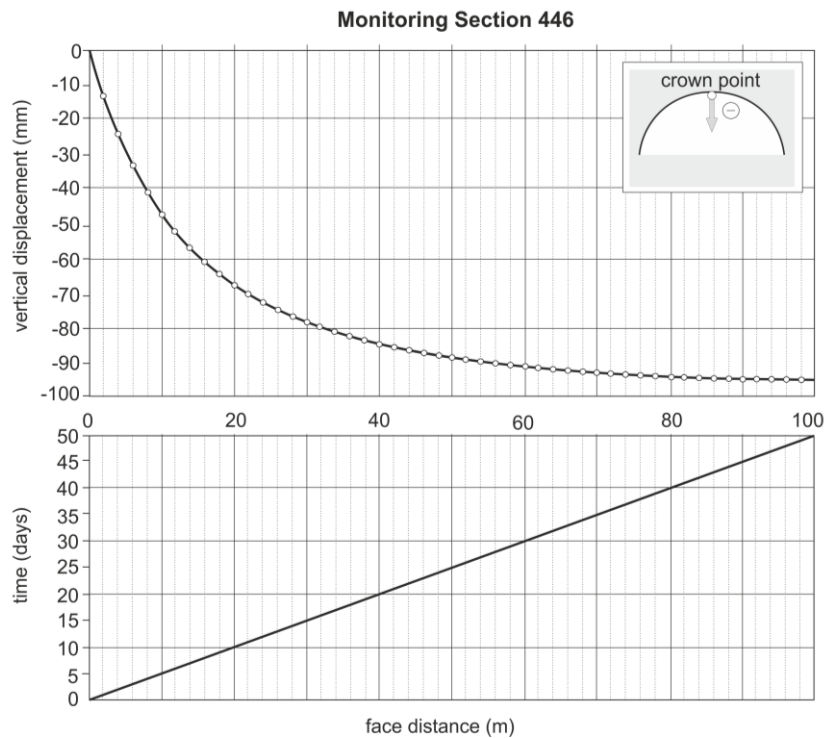


Figure 39. Typical distance - displacement diagram: stabilisation of the top heading advance with continuous face advance

As shown in Figure 38 and 39, there is no qualitative difference between both visualisation approaches and the interpretation of the system behaviour is straightforward. In both diagrams, the displacement rate continuously decreases with time and thus also with the increasing face distance.

### 5.2.3.2. Changing advance rate and construction stops

It is generally accepted that the displacement development is influenced by the stress redistribution in the ground due to the face advance, and time dependent behaviour of the ground and the support. The effect of the face advance dominates the displacement development [15]. Plotting the displacements against the face advance eliminates the discontinuities (kinks) in the displacement development curve (Figure 40), caused by the different advance rates.

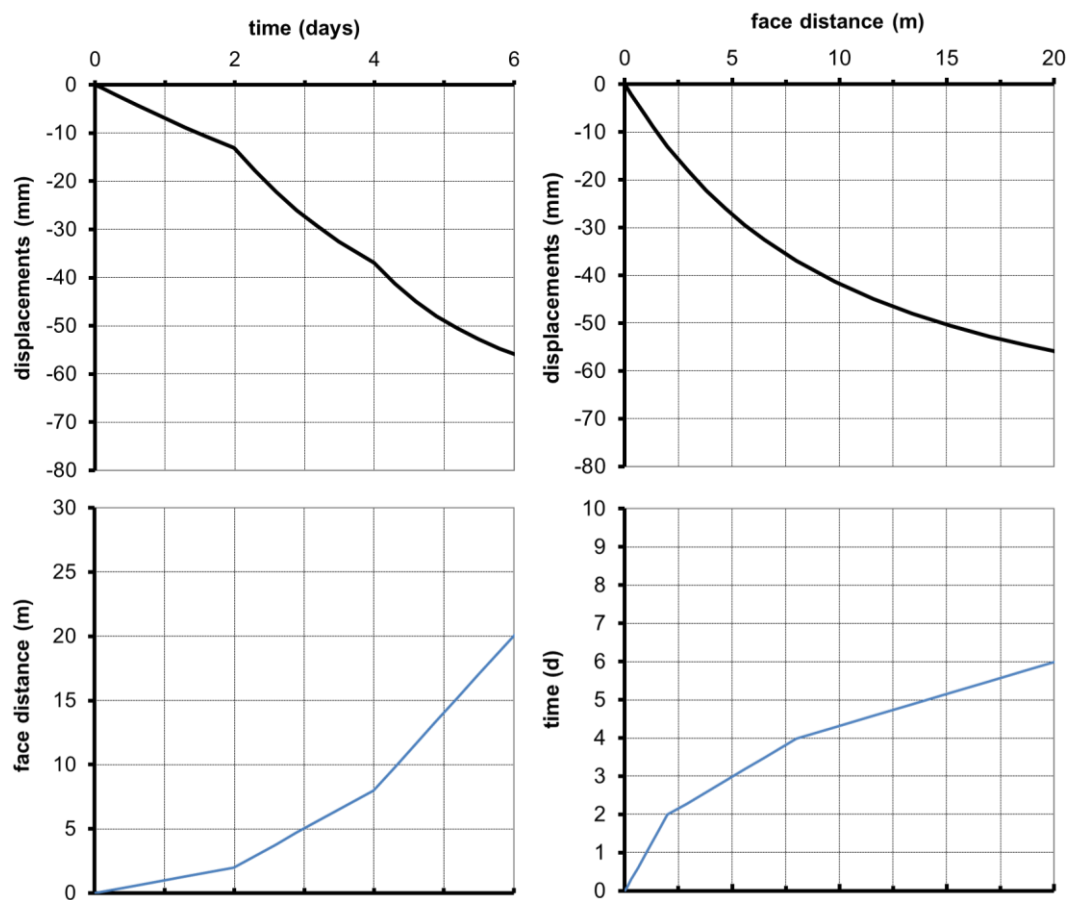


Figure 40. Displacement development in case of unsteady progress [15] displayed in a time-displacement diagram (left) and a distance-displacement diagram (right)

Analogously, a prolonged construction stop causes the displacement development to halt (Figure 41) due to the stop of the advance-related stress redistribution. A plot of the same data in a distance-displacement diagram is shown in Figure 42.

The significant increase of displacements after the restart in the time-displacement diagram may lead to some misinterpretations. Analysing the displacement characteristic in the dis-

tance-displacement diagram eliminates the ambiguity in the interpretation as the discontinuity in the graph nearly vanishes. The main advantage of plotting displacements over face distance is the easier assessment of the stabilization process for excavations with varying advance rate. This kind of representation is meaningful only in case of analysing singular construction phases.

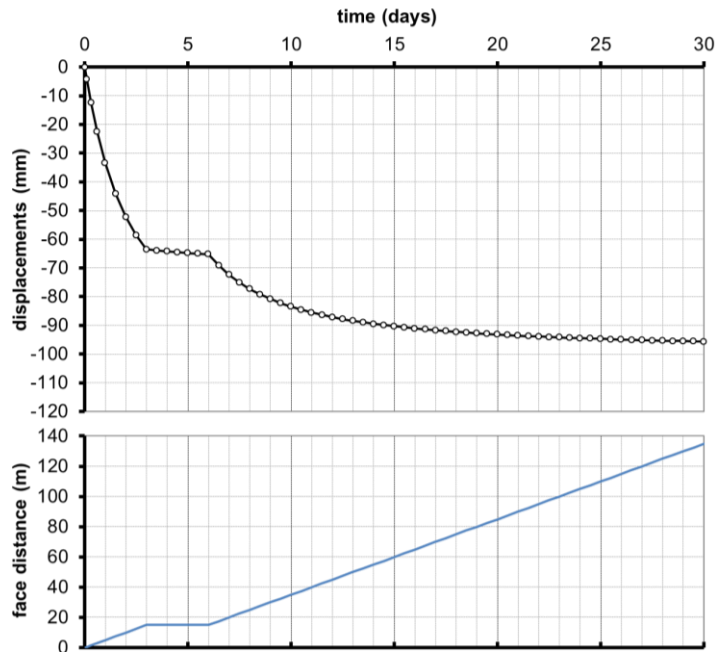


Figure 41. Displacement development in case of a construction stop; time-displacement diagram

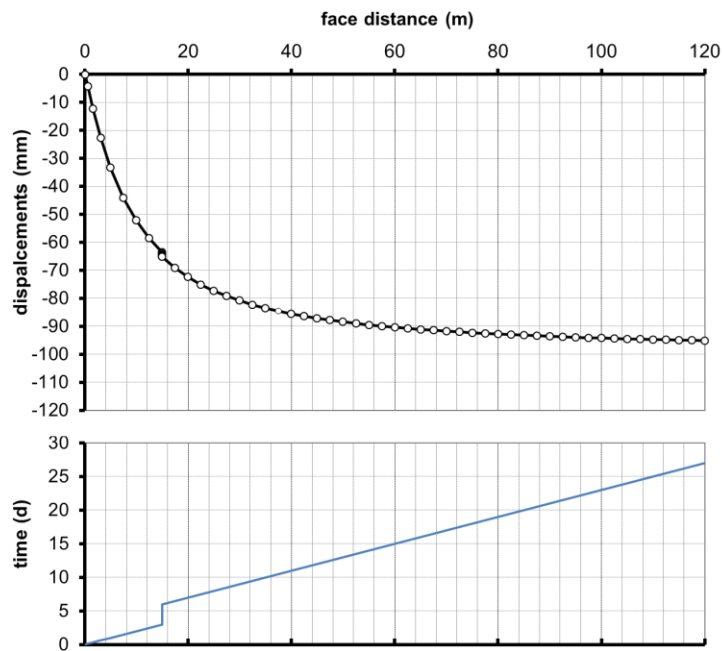


Figure 42. Displacement development in case of a construction stop; distance – displacement diagram

### 5.2.3.3. Construction phases

Subsequent excavation steps, like bench or invert excavation or the excavation of a parallel heading lead to a temporary increase of displacement rates. This influence can be observed up to several diameters ahead of the bench face. The maximum displacement gradient can be observed at the moment of face passage. The assessment of the system behaviour's normality is fairly simple if the distance between the construction phases (e.g. top heading and bench) is sufficiently high and there is no mutual interaction between the stress redistribution processes.

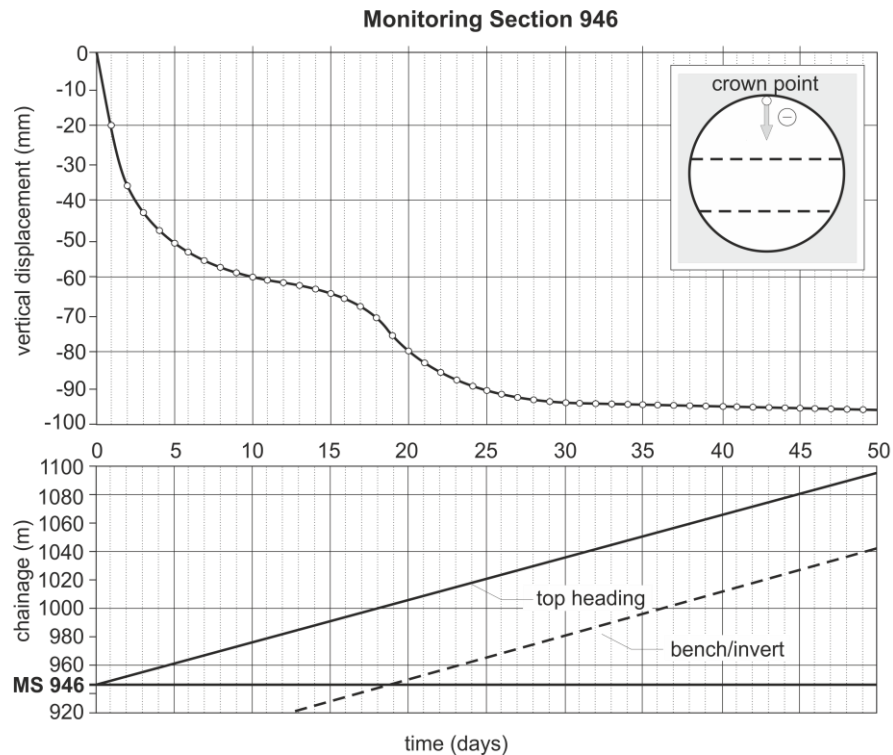


Figure 43. Time - displacement diagram for top-heading and bench excavation

In case of mutual interaction between the construction phases, advanced interpretation methods and prediction models (as described in the following chapters) should be applied.

### 5.2.3.4. Influence of the support installation

Changes in the support resistance have an observable effect on the system behaviour. In particular, the effect of the ring closure on the displacement characteristic is clearly visible (Figure 44). As shown therein, the displacement rate decreases significantly, leading to a “kink” in the time-displacement diagram after the installation of the invert.

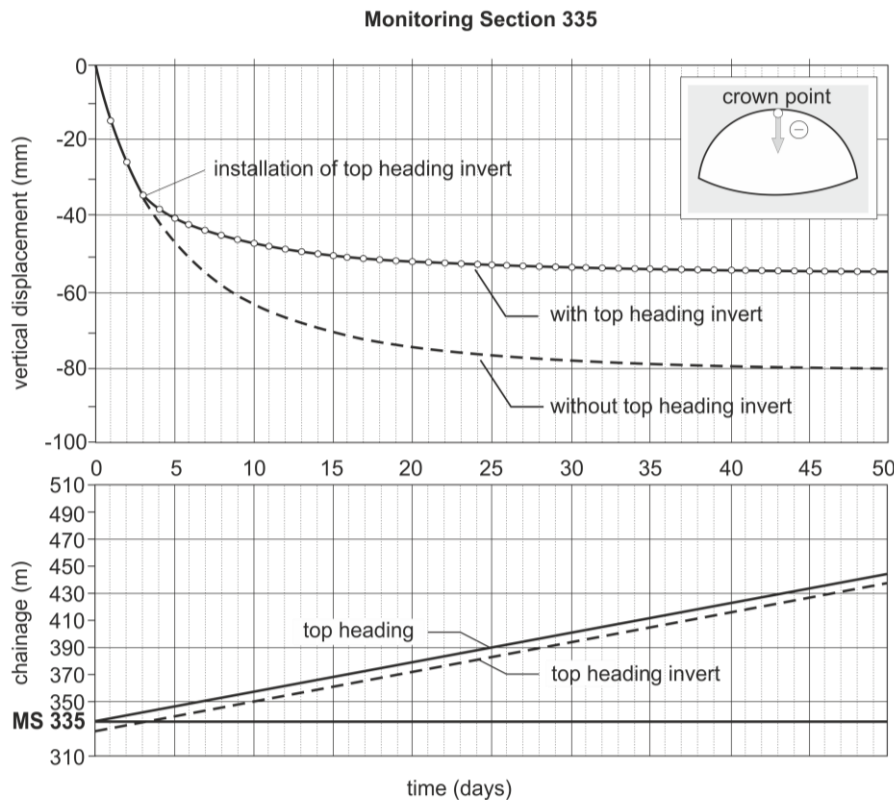


Figure 44. Influence of the installation of a top heading invert on the displacement development

#### 5.2.4. Displacement development prediction model

The work of Sulem, Barlow, Sellner et al. [16, 17, 18, 19, 20] has resulted in development of empirical relationships allowing prediction of the displacement development based on a few readings or previous experience under similar conditions. The effects of the face advance, time, support stiffness and construction phases are combined in one prediction model. The relationship always provides stabilization of the system (displacement rates eventually converge to zero).

The following figures illustrate the process of displacement prediction in comparison to the measured data.

Figure 45 shows the prediction of the displacement development (dashed blue line); based on the zero reading, one consecutive reading, previously observed system behaviour and an assumed construction advance. Displacement development ahead of the face is also displayed.

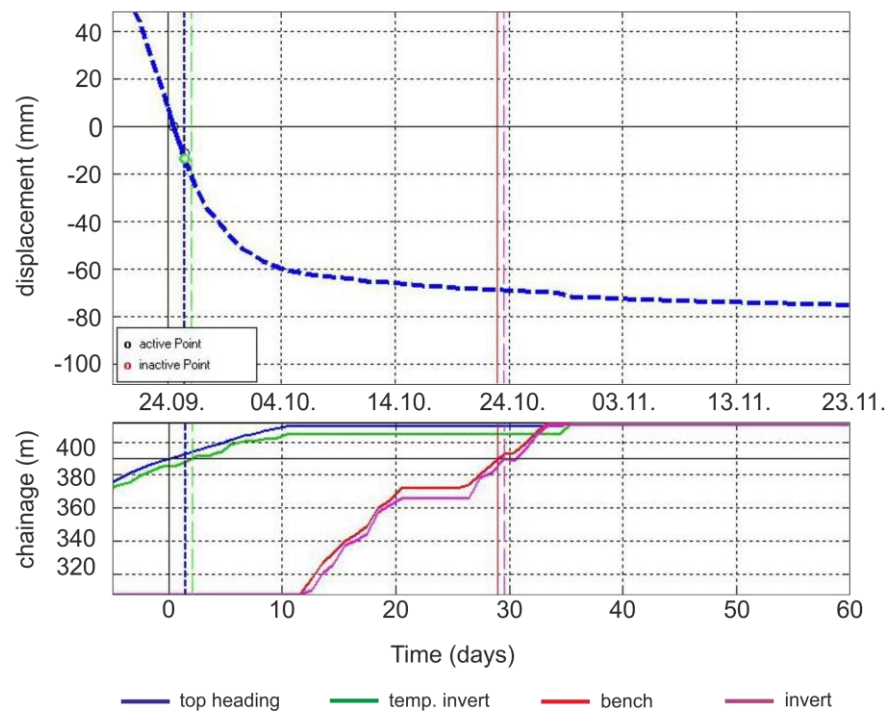


Figure 45. Prediction of the displacement development for the top heading (dashed blue line) based on previously observed system behaviour, zero reading and one consecutive measurement at MS 390 [21]

Figure 46 shows the effect of the installation of a temporary top heading invert at the time of the first consecutive measurement (green line).

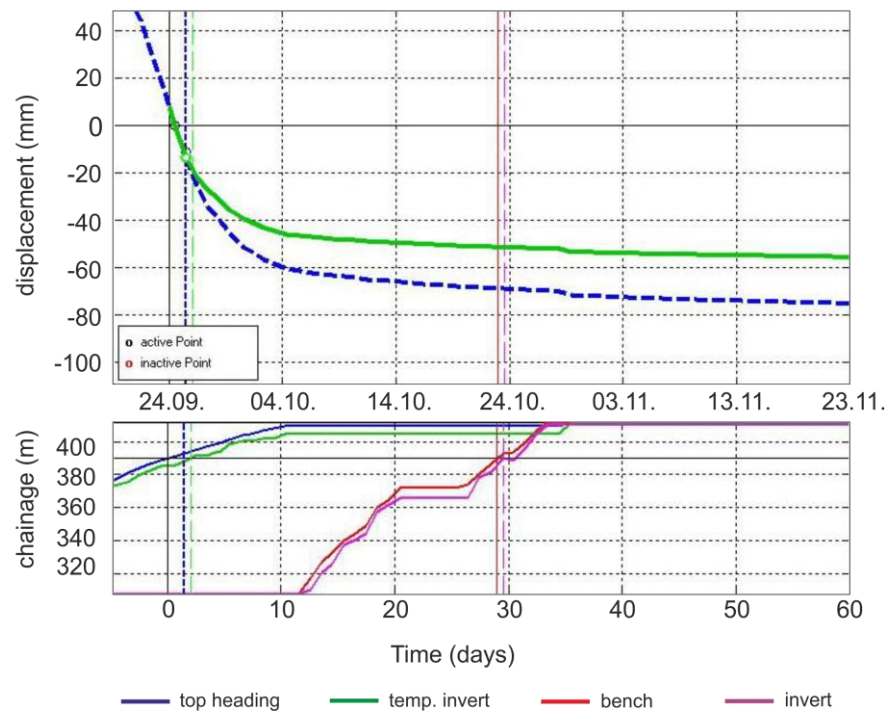


Figure 46. Prediction of the displacement development with temporary top heading invert installation (green line) [21]

Figure 47 shows the prediction of the displacements including bench excavation (red line).

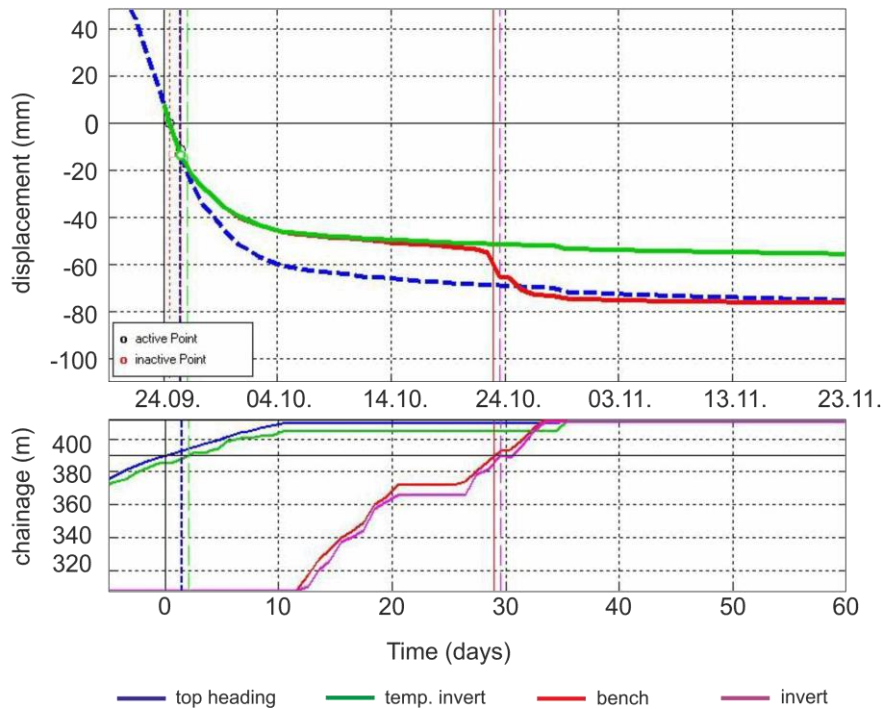


Figure 47. Subsequent prediction of the displacement development due to bench excavation (red line) [21]

Figure 48 displays the comparison between predicted and measured displacements (black circles). As can be seen there is a good agreement between prediction and measured data.

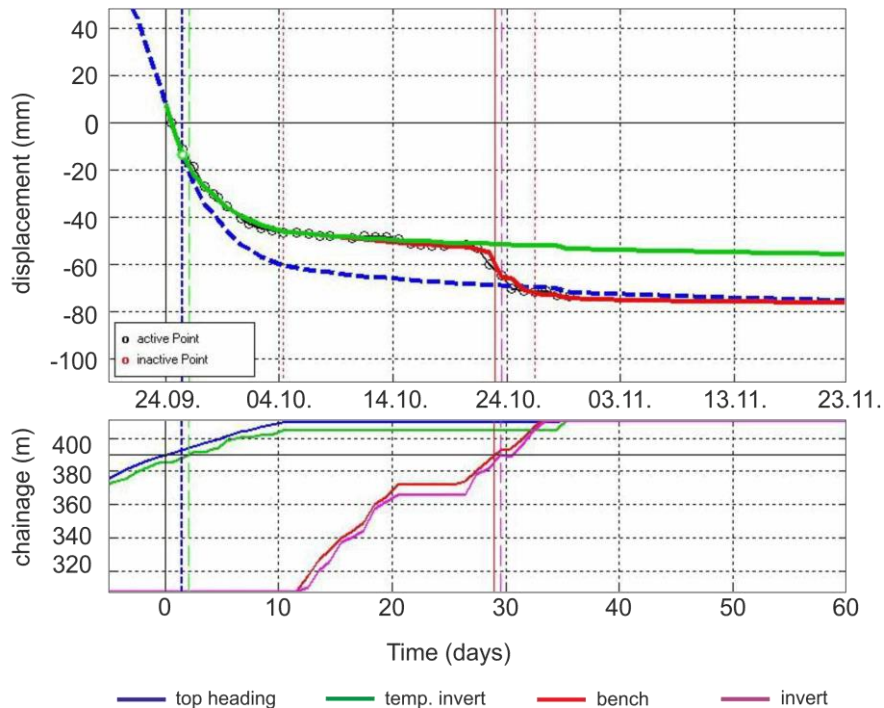


Figure 48. Comparison between the observed and predicted system behaviour [21]

#### 5.2.4.1. Identification of deviations from normal behaviour

Figure 49 shows the development of the measured crown settlements for a top heading excavation with a temporary top heading invert in a tunnel with medium overburden in a fault zone. The predicted displacements without (blue dotted line) and with temporary invert (green line) are displayed to provide the basis for comparison with the actual data. A deviation of the measured displacements to the predicted displacements (on day 40 in the graph) indicates a failure of the temporary invert.

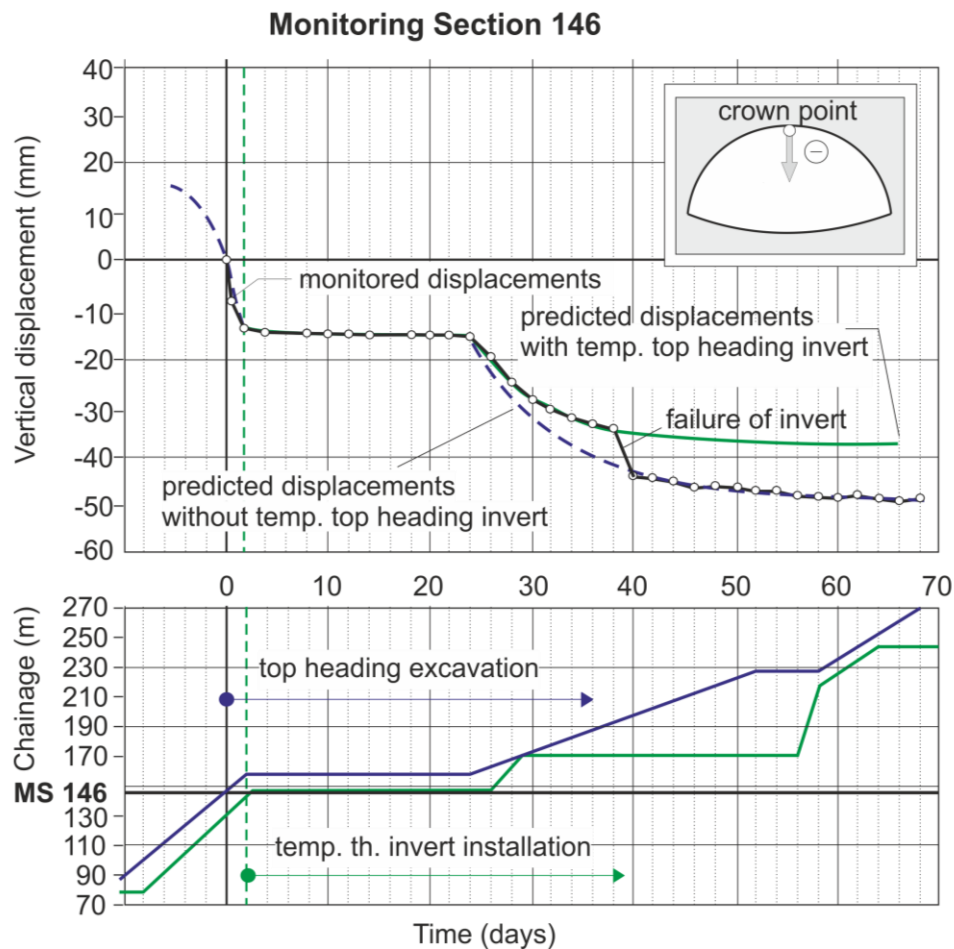


Figure 49. Example of time-displacement plot with measured displacements deviating from the predicted displacements due to a failure of the temporary top heading invert

Further development of the displacements indicates stabilization at the level of the behaviour without temporary top heading invert.

The following example originates from a tunnel with shallow overburden in alluvial soils. Figure 50 shows the development of the vertical displacements of the top heading foot at monitoring section 191 [15]. The relative strong initial settlement is noticeable, indicating a limited stress transfer in the longitudinal direction.

However, after the installation of the temporary top heading, invert settlement rates decreased quickly. The measured displacements after the zero-reading up to the sixth reading corresponded quite exactly to the predicted settlement characteristics. Starting from October

30<sup>th</sup> significant deviations from the predicted behaviour appeared, showing progressive increase in the settlement rates.

Initially the first deviation indicates a failure of the temporary top heading invert. Subsequently, the observed behaviour progressively deviates from the predicted one, with high displacement rate increase. The fresh shotcrete lining could not sustain the quick additional load and failed.

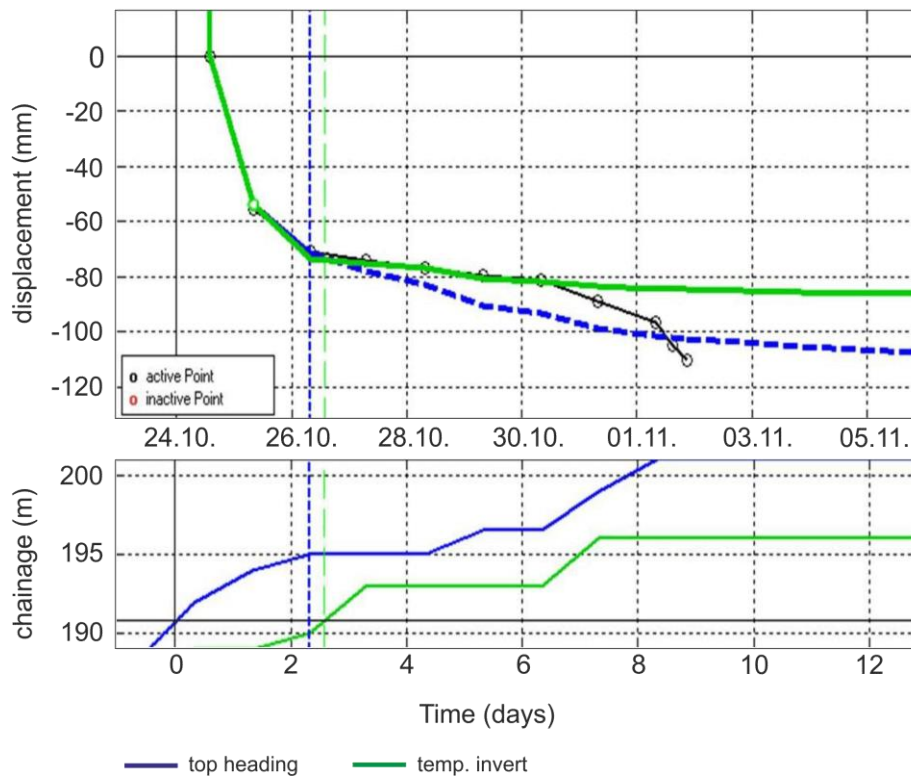


Figure 50. Deviation of system behaviour from the predicted displacement development, leading to collapse of top heading [15, 21]

### 5.2.5. Displacement vectors in longitudinal and cross section

The displacement development is plotted in sections perpendicular and parallel to the tunnel axis (Figure 51). The vector plots (Figure 52, Figure 53) can show the influence of the structure and quality of the surrounding ground, which has a significant influence on the displacement characteristics [22, 23, 24, 25, 26]. In addition, the kinematical aspects of the interaction between the support and the ground can be observed.

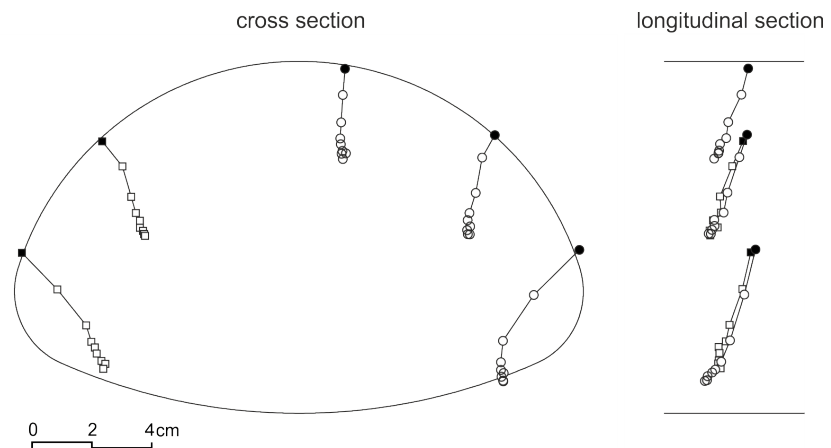


Figure 51. Displacement vector plot in cross- and longitudinal section [41]

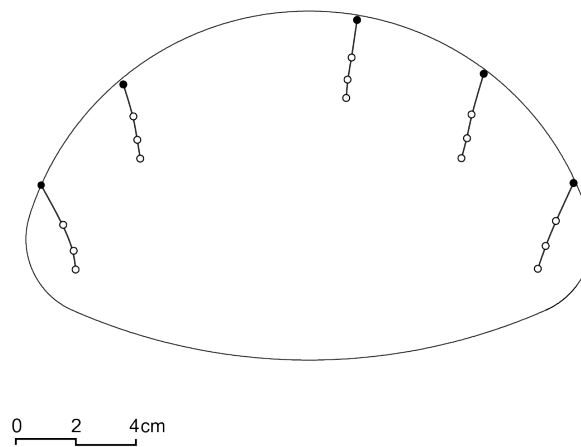


Figure 52. Displacement vector plot for top heading excavation [41]

Figure 53 shows the influence of the temporary top heading invert installation; the displacement rates generally rapidly decrease, and the sidewall points show divergence.

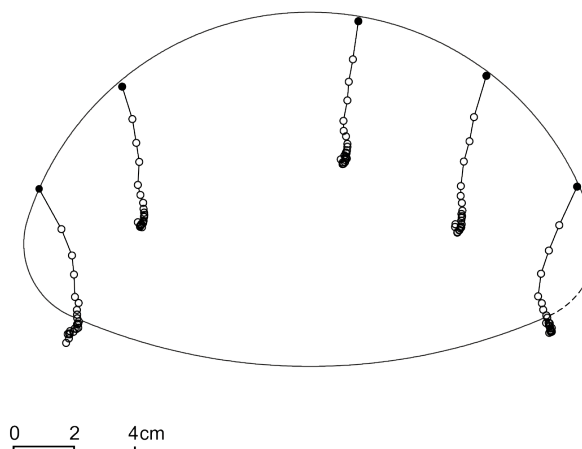


Figure 53. Displacement vector plot; influence of temporary top heading invert [41]

Figure 54 shows an example of the influence of a series of steeply dipping faults, striking sub-parallel to the tunnel axis, on the displacement characteristic.

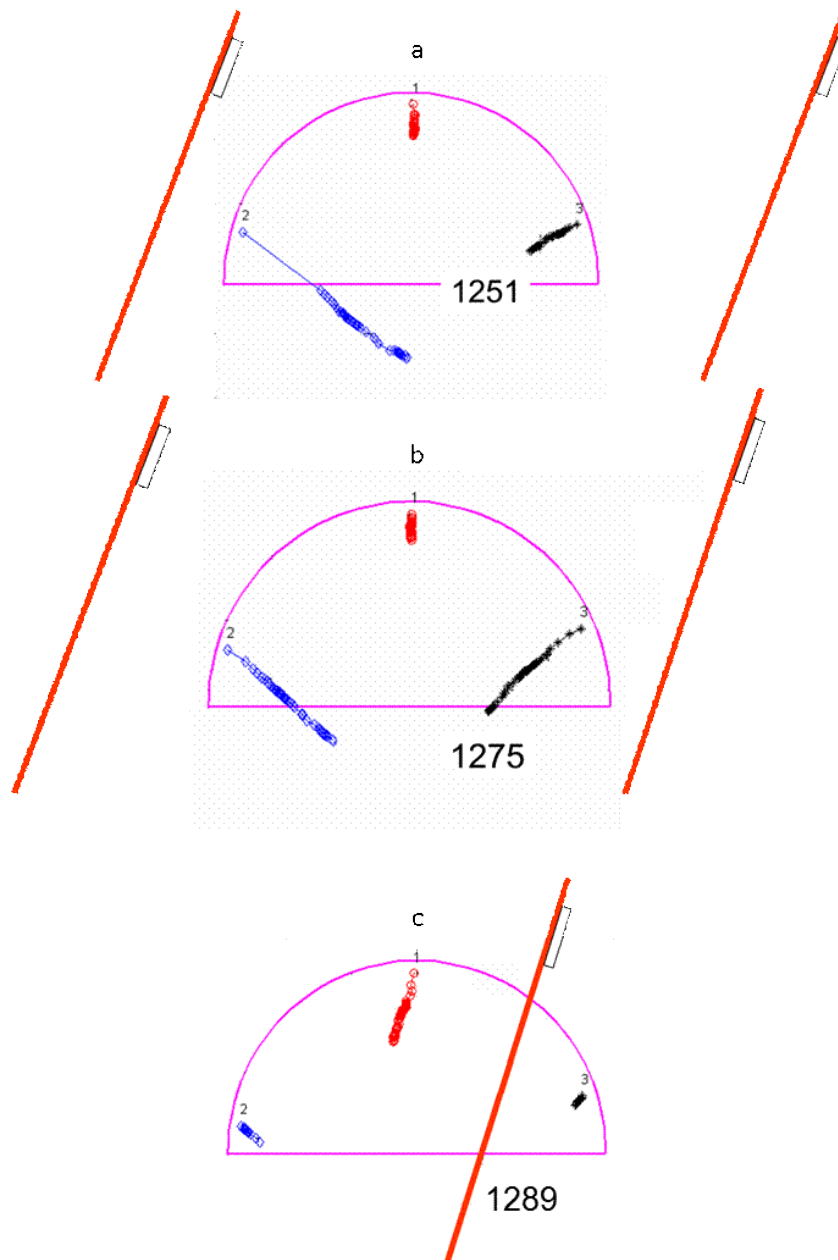


Figure 54. Displacement vector plots; influence of faults outside and inside the excavated cross section; a) stress concentration between left sidewall and fault causes large displacements of left sidewall; b) additional fault close to right sidewall leads to increase of displacements on right sidewall; c) fault zone in excavation profile causes only increased displacements in the crown [27]

### 5.2.6. State line (deflection curve) diagrams and trend lines

State lines are produced by connecting the measured displacements in a line (e.g. crown, shoulder or sidewall) of a number of monitoring points along the tunnel axis at a certain time. A number of state lines over a large section for a specified time span can be shown on one plot.

This allows a good overview on the displacements over a longer stretch and a longer time period. Single displacement components (like vertical, horizontal or longitudinal displacements) are shown [28, 29].

In order to eliminate the influence of different times/distances of the zero reading to the face position/excavation time, it is recommended to consider the displacements at the face to be zero. This requires an estimation of the displacements between the face and the monitoring section. This either can be done by extrapolating the spline to the face position, considering the development of displacements of minimum three previous monitoring sections. The second and more appropriate method is to use the prediction model [19, 21] and calculate the displacement between monitoring section and face.

Figure 55 shows the influence of each excavation step on previously excavated and supported tunnel sections.

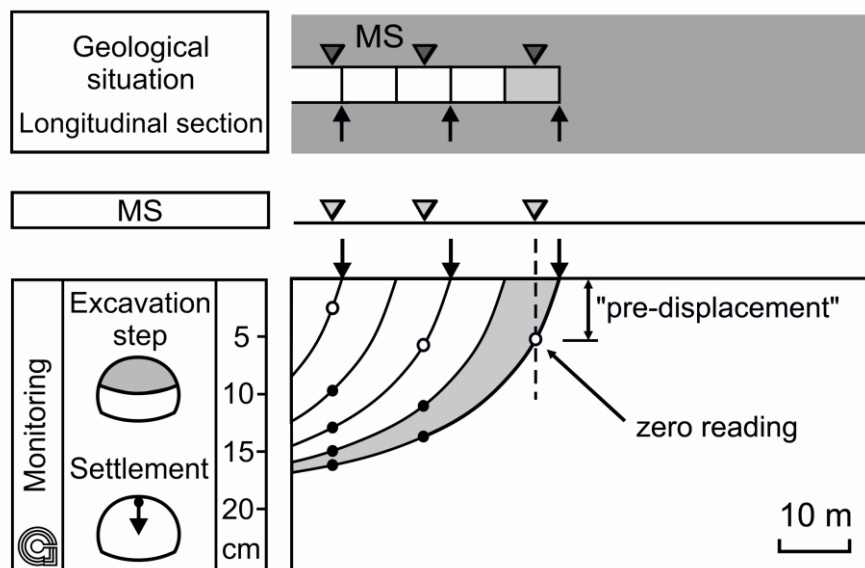


Figure 55. Deflection curve diagram for crown settlements [28, 30]

Deflection curves are usually combined with a trend line. Trend lines are created by connecting values from the deflection curves at a constant distance behind the face (Figure 56).

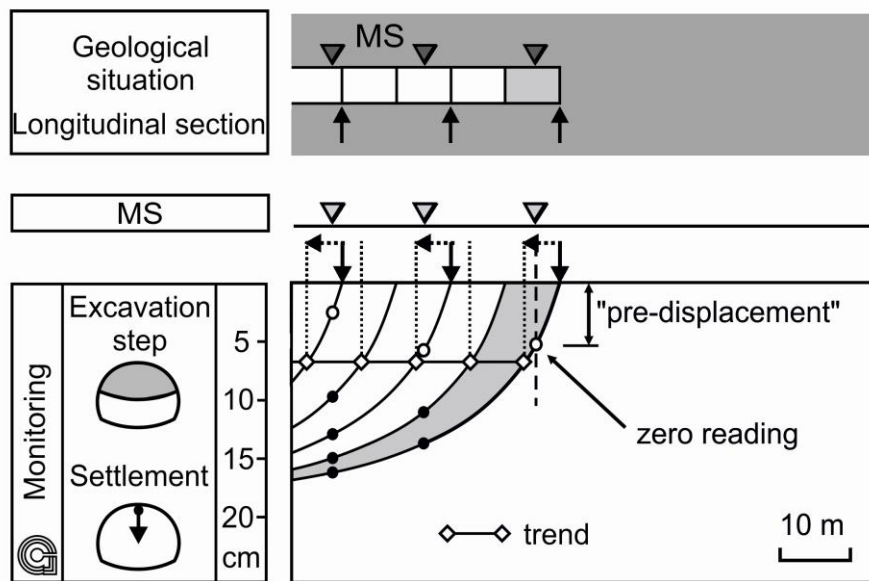


Figure 56. Deflection curve diagram with trend line [28, 30]

"Onion-shell-type" deflection curves and a horizontal trend line development are obtained when tunnelling in homogeneous ground conditions and same boundary conditions.

Figure 57 shows the influence of a fault zone, crossing the tunnel axis approximately perpendicularly. Displacements increase, as the tunnel approaches the fault zone. The influence on sections further away from the fault zone can be also seen. Once the face has passed the fault zone, the influence on sections "behind" the fault zone should be minimal.

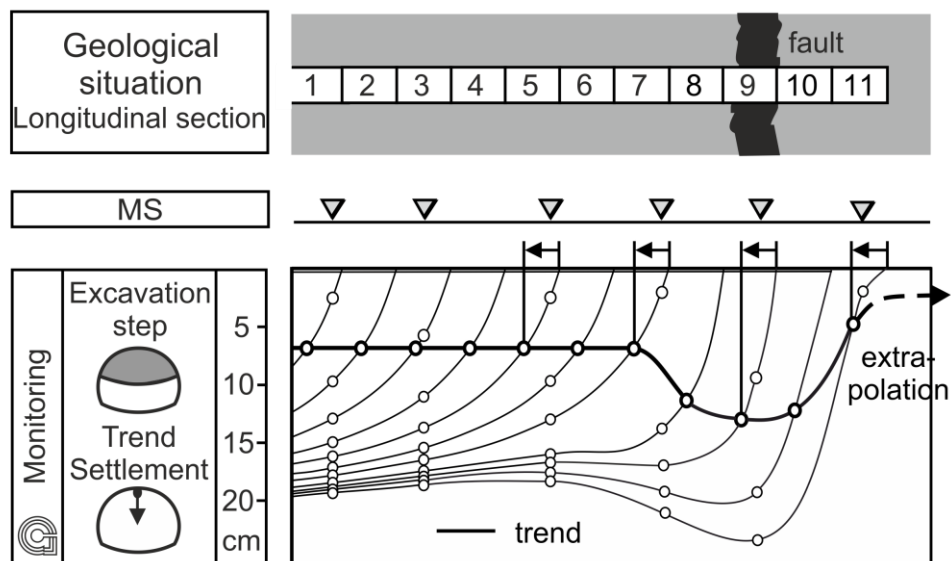


Figure 57. Deflection curve diagram with trend line in case of passing through a fault zone [30]

### 5.2.7. Evaluation of displacement differences or ratios

Evaluating the ratio between displacement components or differences and plotting them as a trend can help in detecting changing system behaviours or changing ground conditions outside the excavation area. There are different possibilities to combine displacement components in each section developing along the tunnel axis:

- Same component in different monitoring points
- Different components in one monitoring point

#### 5.2.7.1. Same component in different monitoring points

In cases, where the absolute difference of displacements can be relevant for the system behaviour (for example stresses in the lining) it is recommended to evaluate displacement differences between different points (e.g. surface settlement versus crown settlement Figure 58, crown versus sidewall Figure 59). Figure 58 shows a situation where low confinement in the ground caused a shear failure, leading to a higher surface than crown settlement.

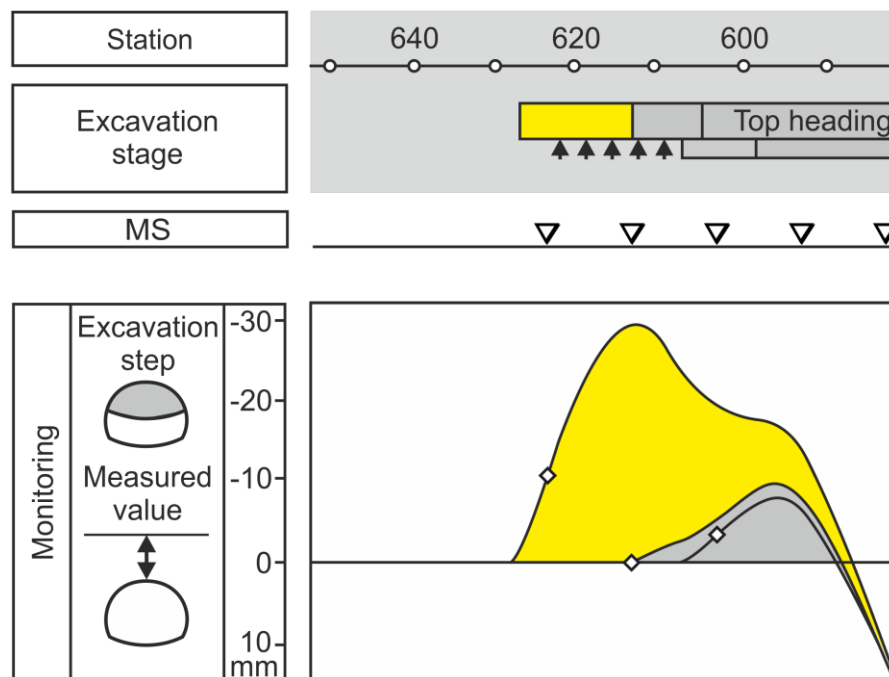


Figure 58. Evaluation of settlement difference between surface and crown [29]

Higher sidewall than crown settlements can indicate pronounced shear displacement between ground and lining, or a failure of a temporary top heading invert. The change in the displacement trend in Figure 59 indicates an arching over a weak zone in the ground. This results in a higher loading of the support in the area, where the temporary invert was already installed, while in the front section a failure in the ground leads to relatively high sidewall settlements.

During the top heading excavation the system stabilized, however during bench excavation a daylighting collapse occurred in this section. The changing trend of the horizontal displacements in this section is shown in Figure 60.

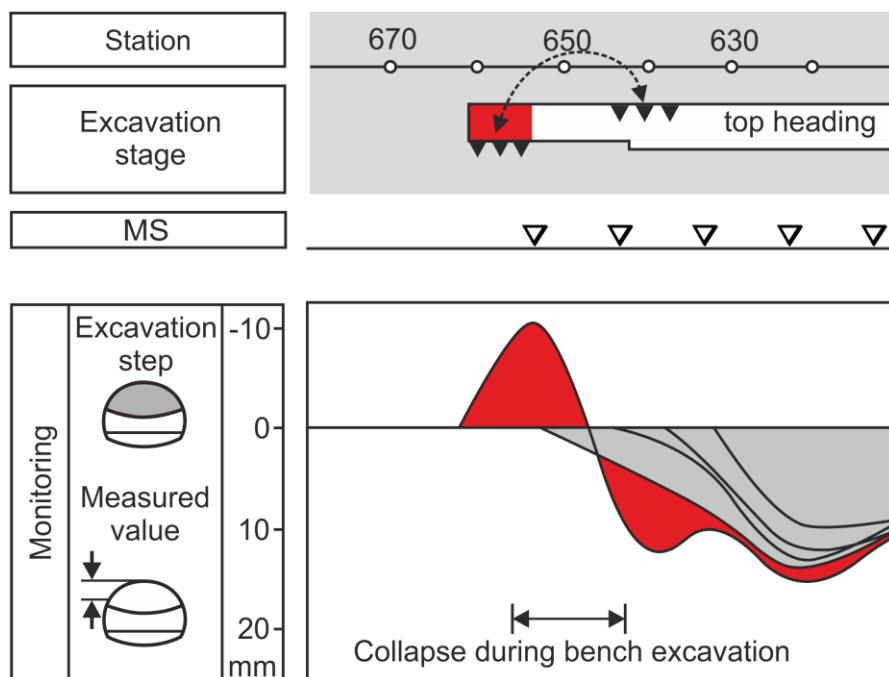


Figure 59. Development of settlement difference between crown and left sidewall [modified after 29]

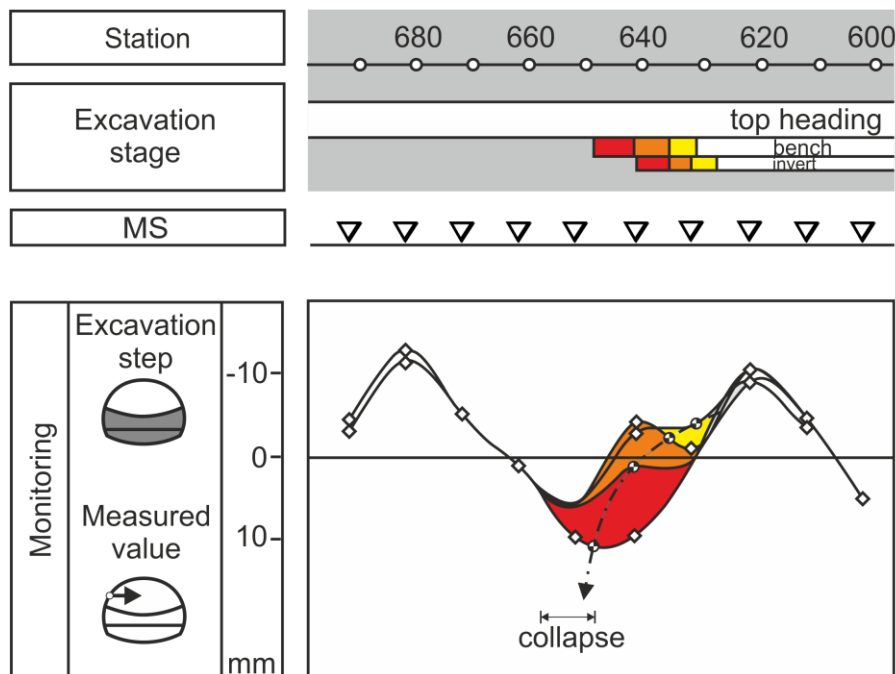


Figure 60. Development of horizontal displacements and trend of the left sidewall during bench and invert excavation [29]

Ratios of displacement components can be used with single components of different monitoring sections (e.g. ratio of crown settlements versus sidewall). In homogeneous material, the

crown settlement will generally be higher than the sidewall settlement. When approaching a steeply dipping fault, which crosses the tunnel axis in an acute angle, larger settlements of the sidewall than of the crown will indicate an overloading of the ground between sidewall and fault. As shown in Figure 61, the ratio of the left sidewall to crown settlement considerably increases well before the fault is visible at the face, allowing time to take the proper action. With further advance of the excavation face, the ratio of left sidewall to crown settlement reverts to "normal".

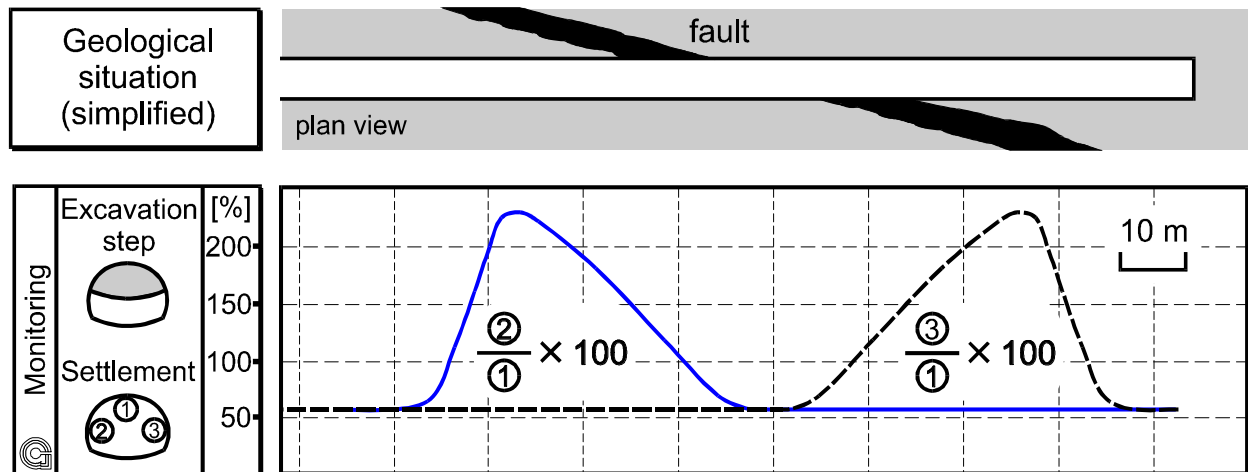


Figure 61. Trend lines showing the ratio of the settlements left and right sidewalls to the crown [30]

#### 5.2.7.2. Different components in one monitoring point

The displacement vector orientation L/S is the ratio between longitudinal displacements (L) and settlements (S) and is expressed in terms of an angular deviation of the displacement vector from vertical [31, 32, 33, 30, 34].

A rotation of this displacement vector against the direction of excavation indicates relatively soft ground ahead of the face. In contrast, a rotation in direction of the excavation reflects relatively stiff conditions ahead. Figure 62 shows the basic development of the vector orientation trend L/S when tunnelling through a zone of relatively soft ground.

The displacement vector trend L/S (in this case for the crown point) deviates from a "normal" orientation against the direction of excavation when approaching the zone of soft ground. The "normal" vector orientation strongly depends on the ground structure and stress conditions and has to be determined separately for each condition [35, 36, 37, 38].

After entering the soft zone, the vector tends to return to the "normal" orientation. When approaching the stiff ground, an inverse tendency can be observed. The displacement vector deviates in direction of excavation and reaches a maximum at the transition. After entering the stiff zone, the vector returns to normal.

The L/S trend evaluation is the most appropriate method to identify changes in the ground conditions.

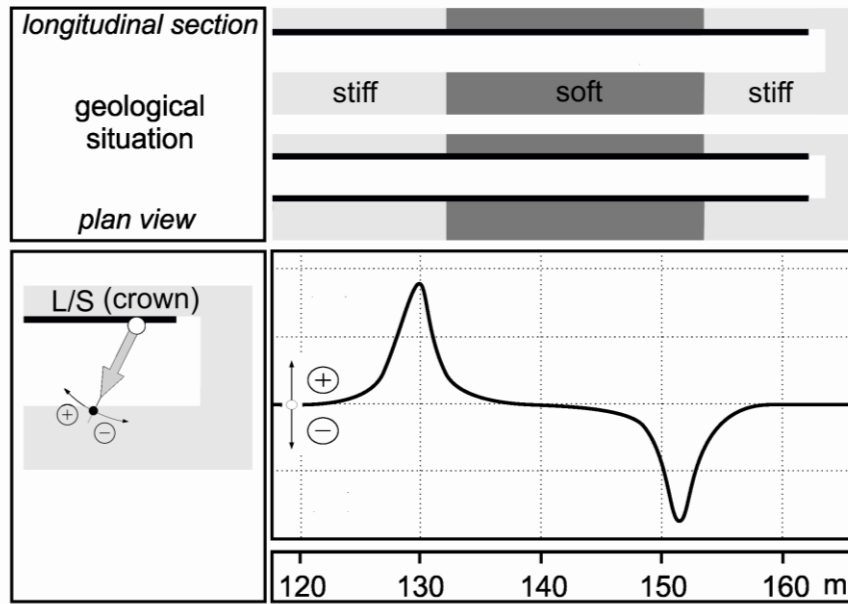


Figure 62. Example for displacement vector orientation trend  $L/S$  [39]

In order to illustrate the principles presented above, a case study with their application is briefly presented (Figure 63, Figure 64). The upper diagram in Figure 63 shows the development of crown settlements of a tunnel section of around 120 m length [15, 40]. The trend line of the ratio of longitudinal displacements and settlements ( $L/S$  – displacement vector orientation, evaluated as an angle) is illustrated at the bottom. The settlement curves show a nearly constant development with a small variation. In contrast, the development of the displacement vector orientation clearly shows a significant deviation from a certain “normal” range against the direction of excavation, beginning at approximately 40 m (three tunnel diameters) before the fault zone. This tendency indicates changing ground conditions with weaker ground expected ahead of the face. In this case, the range of normal displacement vector orientations is between  $4^\circ$  to  $8^\circ$  against the excavation direction.

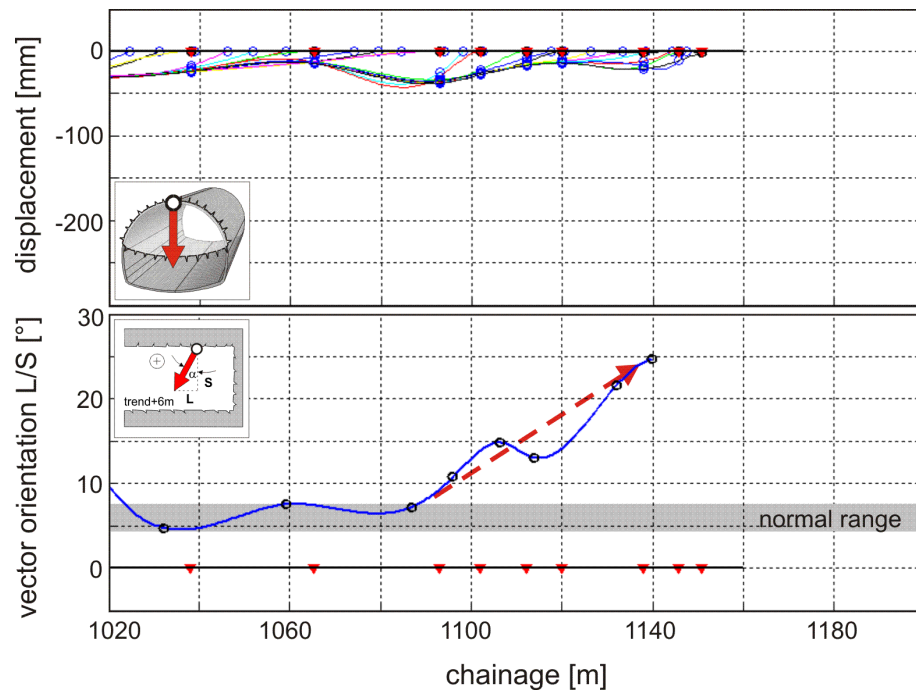


Figure 63. Deflection curves of vertical displacements of the crown (top) and trend of ratio between longitudinal and vertical displacements (below). Deviation of displacement vector orientation indicating change of ground quality ahead of face [40]

Approximately half a tunnel diameter ahead of the fault zone, the L/S vector orientation reaches its peak value (Figure 63). After entering the fault zone, it remains oriented against the direction of the excavation and gradually decreases to the “normal range” (Figure 64).

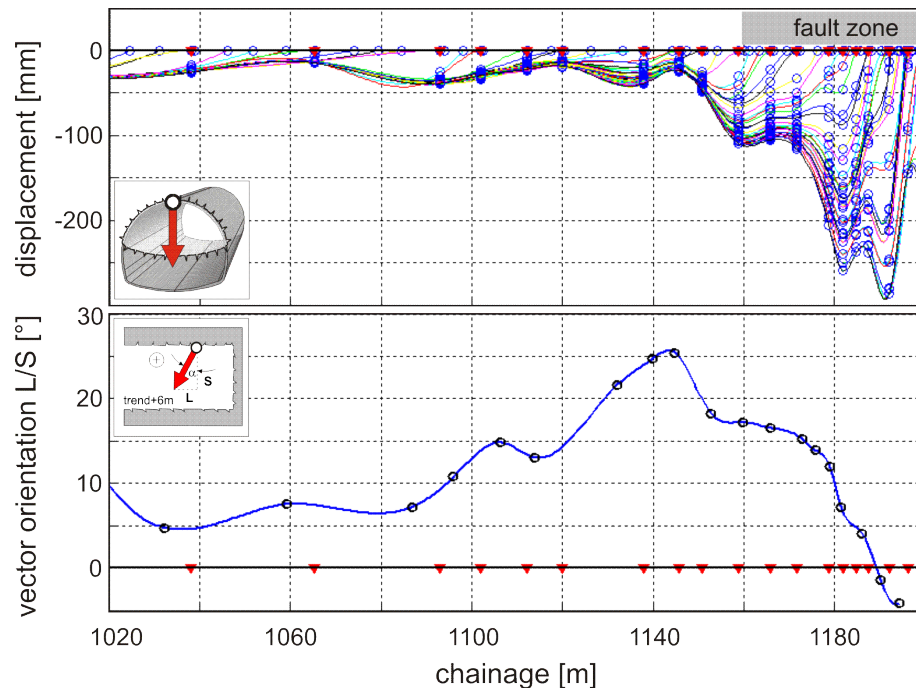


Figure 64. Deflection curves showing the increased displacements in the fault zone, encountered at around chainage 1160, as well as the trend of the displacement vector orientation [15, 40]

By approaching the end of the fault zone, the vector orientation changes towards the direction of excavation, thus indicating more competent ground ahead. Having entered competent ground and with sufficient distance between the face and the “transition zone”, the vector orientation returns to normal again.

#### **5.2.7.3. *Combination of different trends***

To determine the spatial orientation of regions with different ground quality ahead, interpretation of the combination of different displacement trends can be used [41, 42]. Depending on the relative spatial orientation of the transition, characteristic changes in the trends of the monitoring results of different points can be observed.

Figure 65 shows a set of trends for a tunnel excavation through a fault zone. Shown are the characteristic developments of the crown settlements, the vector orientations  $L/S$  (ratio of longitudinal displacements and settlements) and  $H/S$  (ratio of horizontal displacements and settlements) for the crown as well as the vertical displacement ratio between the left and the right sidewall,  $SL/SR$ . While the radial displacements (for example  $S$ ) show a significant change close to the transition, the displacement vector orientations change ( $L/S$ ,  $H/S$ ) already some distance ahead of the transition. Also the displacement ratio of the sidewalls ( $SL/SR$ ) changes significantly when the excavation approaches the weak (soft) zone.

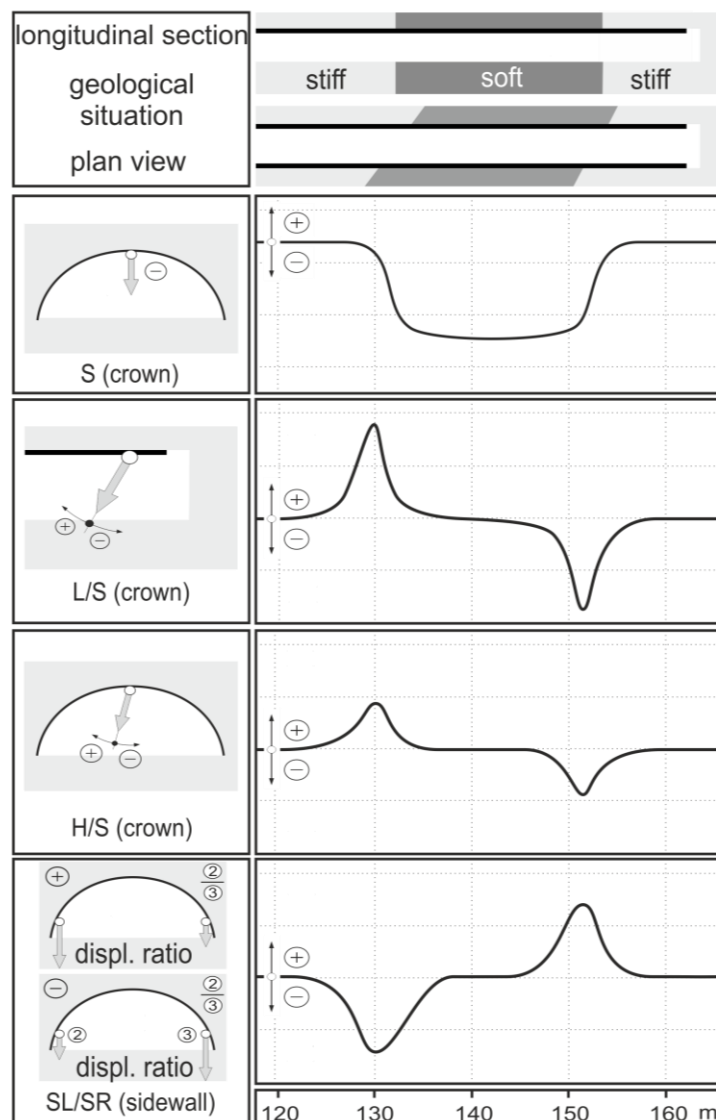


Figure 65. Typical displacement trend lines when tunnelling through a fault zone [41, 42]

### 5.2.8. Displacement vector orientation plot in stereonet

Stereographic projection can be used to display the orientation in space of all points monitored in one measuring section [30, 43]. This facilitates an easy recognition of zones with different stiffness and their spatial orientation. Displayed in Figure 66 is a sequence of vector orientations over three measuring sections of a top heading excavation with three measuring points, approaching rock mass with higher stiffness, striking in an angle of approximately  $45^\circ$  to the tunnel axis. In the first measuring section, crown and sidewalls show displacement vector orientations against the direction of excavation. This deformation pattern is considered to be "normal". With decreasing distance to the relatively "stiff" zone, the orientations of all three vectors change in a characteristic way. The displacement vector orientation of the point closest to the transition shows a significant deviation from the normal orientation, pointing in direc-

tion of excavation. The crown point shows a similar trend, but in a less pronounced magnitude, while the opposite sidewall point shows this trend only to a minor extent.

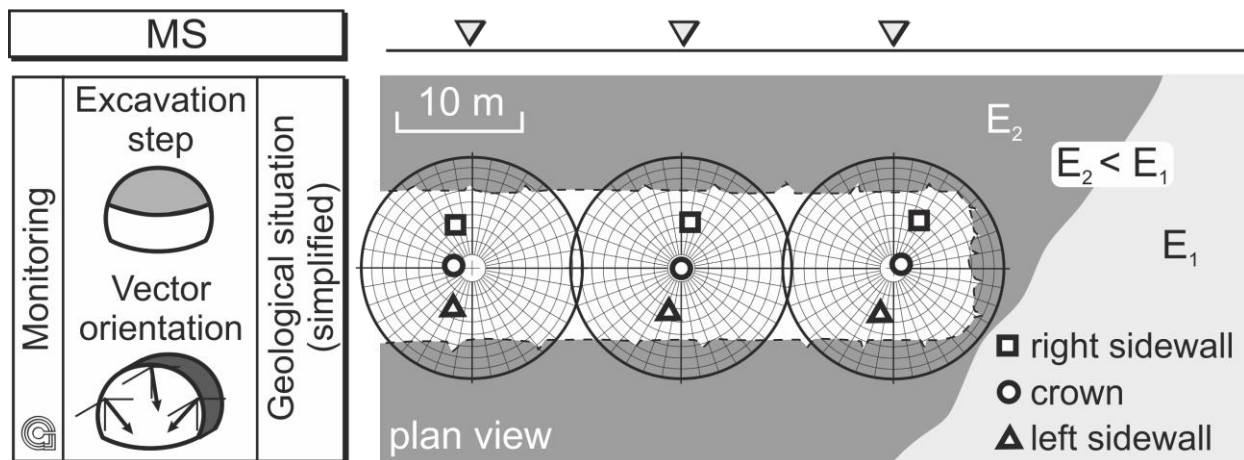


Figure 66. Display of displacement vector orientations in stereoplot (lower hemisphere, note that the vectors of the left sidewall plot on the right side, and vice versa) [30]

### 5.2.9. Stress intensity in a shotcrete lining

The measured displacements can also be used to evaluate the strains in the lining. Considering the time dependent development of the shotcrete properties, the stresses in the lining are calculated. Currently the stresses in the lining are evaluated either using the flow rate method [44, 45] or the hybrid method [46, 47, 48, 49, 50, 51]. The flow-rate method uses a semi-empirical set of relationships describing rheological behaviour of the shotcrete, while the hybrid approach uses a thermo-chemical-mechanical material model. Both approaches require special tests in order to calibrate the model parameters (calorimeter tests, uniaxial compression tests with E-modulus determination, creep and shrink tests and adiabatic tests). The utilisation degree is defined by the distance between actual stress level and the yield surface of the shotcrete at the relevant age [52, 53, 54]. With respect to the evaluation of lining utilisation reference is made also to [55]. The results can be displayed in relation to time, face position or as contour plots.

The example shown in Figure 67 is from a tunnel with shallow overburden in an urban environment excavated with side wall galleries followed by core excavation. The evaluation provides a quick overview of the utilisation of the shotcrete lining.

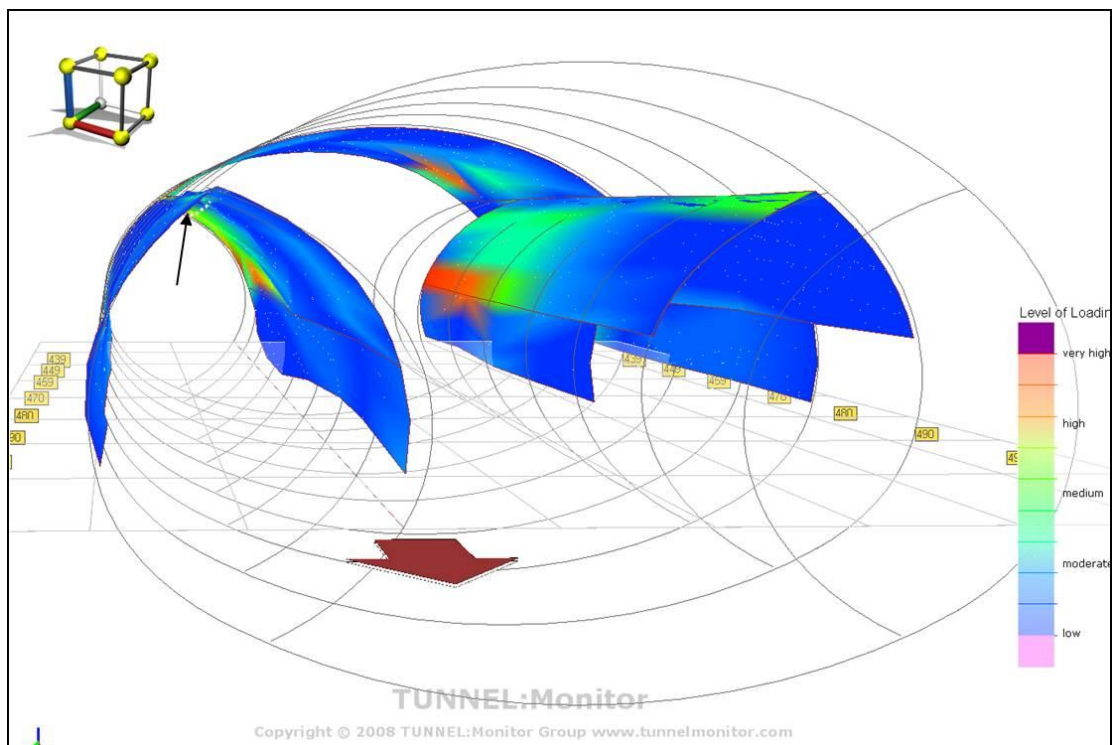


Figure 67. Example of the distribution of the level of loading in the shotcrete lining at a certain time [40, 53, 56]; Tunnel driven with side wall galleries and core excavation (note: red colour indicates very high load level; blue colour indicates a low load level) [57]

#### 5.2.10. Surface displacements

Surface displacements can be displayed in various ways, for example versus time, in cross- or longitudinal sections, as state diagrams, trends, or as contour plots. Figure 68 shows an example of a contour plot of surface settlements in a plan view. Positions of the face of each construction step are shown.

The evaluation gives a good impression of the influence of the construction progress on the distribution and amount of displacements. The maximum of differential settlements (for buildings, tracks, pipe lines, etc.) corresponding to exceedance of defined target values can be directly displayed [56].

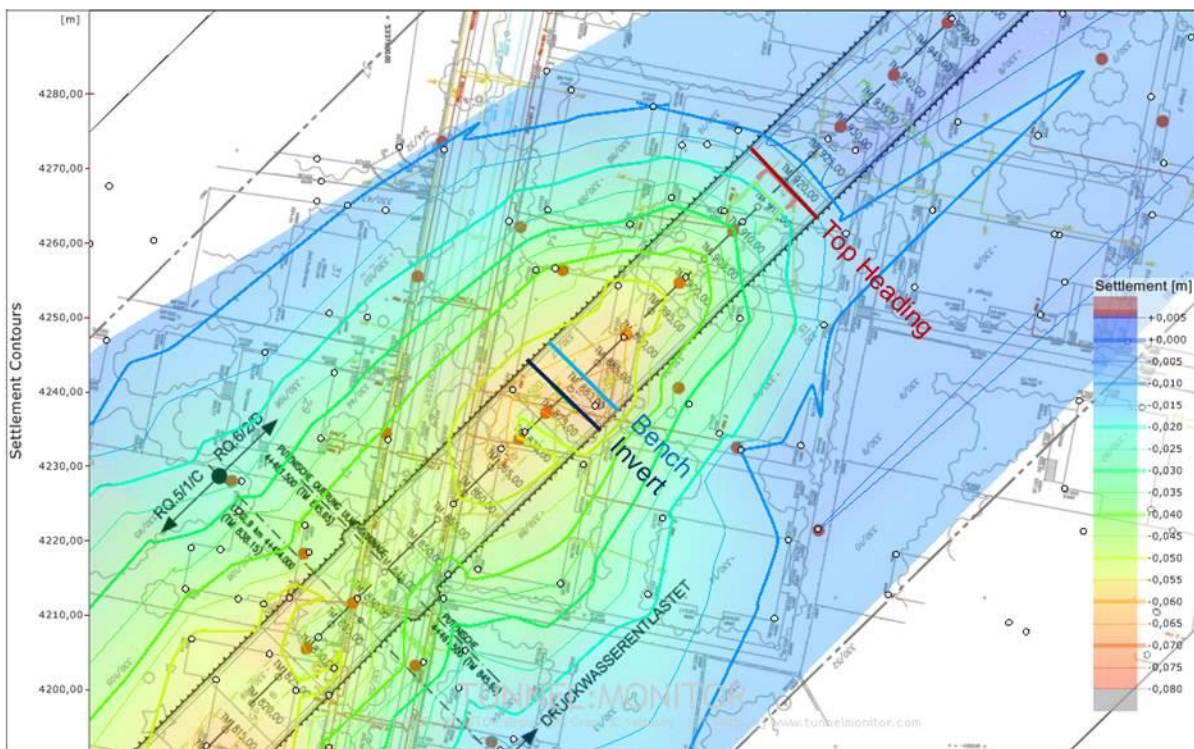


Figure 68. Example of a contour plot of surface settlements with position of top heading, bench and invert faces [57]

### 5.2.11. Value of evaluation methods

Different types of plots and evaluation methods have been presented in this chapter. Table 5 summarises the applicability of the various evaluation methods with regard to specific questions. The time-displacement and distance-displacement plots, for example, are suitable methods for the evaluation and assessment of the stabilisation process, while deflection curves and trend lines are of limited value for this purpose. The latter methods are valuable in assessing the spatial displacement development. Displacement vector orientations in the longitudinal direction as well as 3D orientations are appropriate methods for predicting ground conditions ahead of the face. Utilising this matrix guides the user in choosing the appropriate evaluation method.

Table 5. Value of evaluation methods with regard to specific questions [58]

Value of evaluation methods  ■ very valuable □ limited value - no value	Evaluation target					
	Stabilisation process	Prediction of displacements	Stress distribution longitudinal	Detection of weak zones outside profile; kinematics	Prediction ahead of face	Estimate of lining stress intensity
Time-displacement	■	■	-	□	-	■
Distance-displacement	■	■	-	□	-	■
Deflection lines, trends	□	-	■	□	□	-
Trends of relative displacement values	-	-	-	■	-	-
Vectors in cross section	-	-	-	■	-	■
Vectors in longitudinal section	-	-	■	■	■	-
Spatial vector orientation	-	-	■	■	■	-
Lining utilisation plot	-	-	-	□	-	■
Surface settlement plot	□	-	□	□	□	-

### 5.3. Extensometers

Results of extensometer readings can be displayed versus time, construction progress, relative displacements between two points, strains, or (in case of parallel installation of geotechnical monitoring points at the extensometer head) as radial displacements of the extensometer endpoints. The following evaluations of the measurement data, along with the respective visualisations, should be performed on regular basis:

- Elongation vs. time
- Interval strain between anchoring points
- Determination of absolute displacements by combining the measurements with absolute displacement data (measuring point installed at the extensometer head)

Figure 69 shows an example of a section instrumented with three six-point extensometers. The strains in different periods in each section are displayed in different colours. The extensometers show a pronounced local strain (elongation) in single measuring sections, while the remaining intervals show practically no displacements. The target displacements, combined with the extensometer results indicate shearing along the faults, situated close to the tunnel perimeter. The interpreted shearing along the faults is shown by red arrows. The measurement results allow optimisation of bolt positions and lengths.

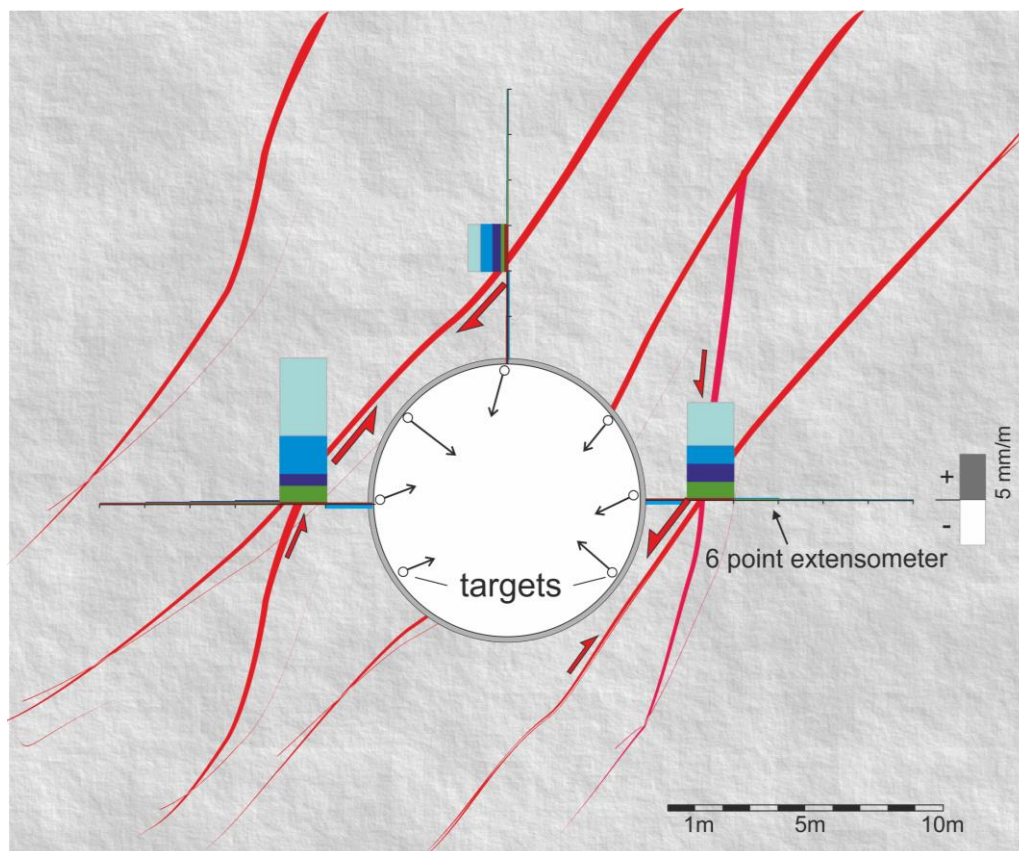


Figure 69. Measuring section, equipped with extensometers; coloured bars show strains per measuring section in subsequent measuring periods

## 5.4. Piezometers

Results of piezometer readings are displayed versus time or/and construction process. Figure 70 shows an example of water pressure reduction when a tunnel approaches and passes the piezometer, installed from the surface to a depth of 42 m at chainage 1146. The ground consists of siltstone/claystone. As can be seen there is a strong drop of about 2 bar (from approximately 3.6 bar to 1.6 bar) within a few days as the tunnel was excavated at the level of the piezometers monitoring section. In the following weeks and with further tunnel progress the water pressure stabilizes at approximately 1.2 bar. Evaluation of water pressure measurements can be helpful in assessing whether drainage measures ahead of the face are required.

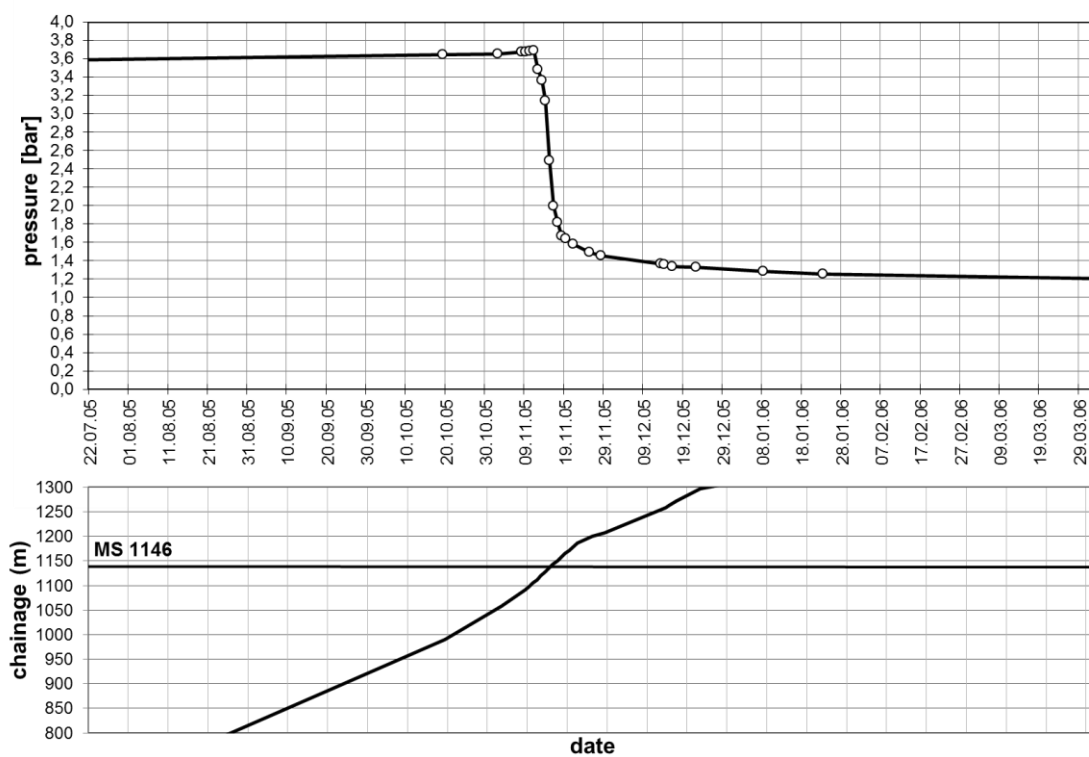
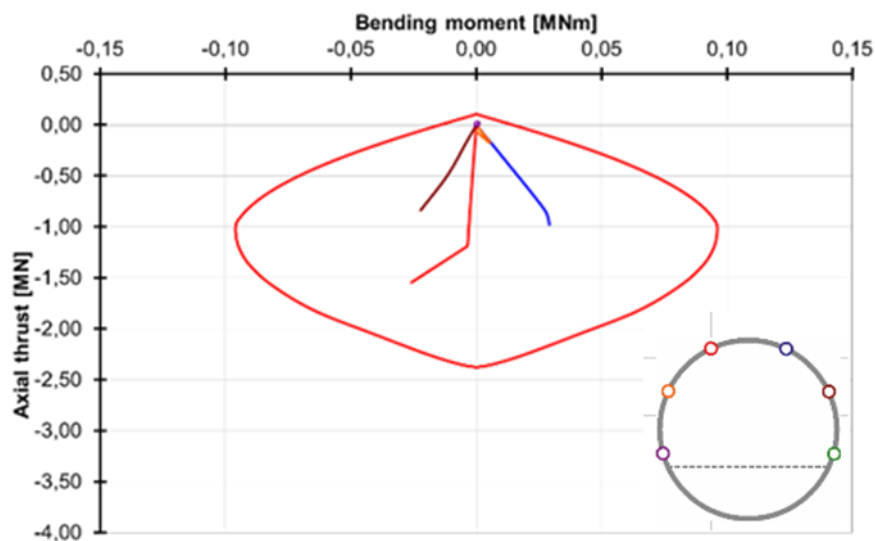


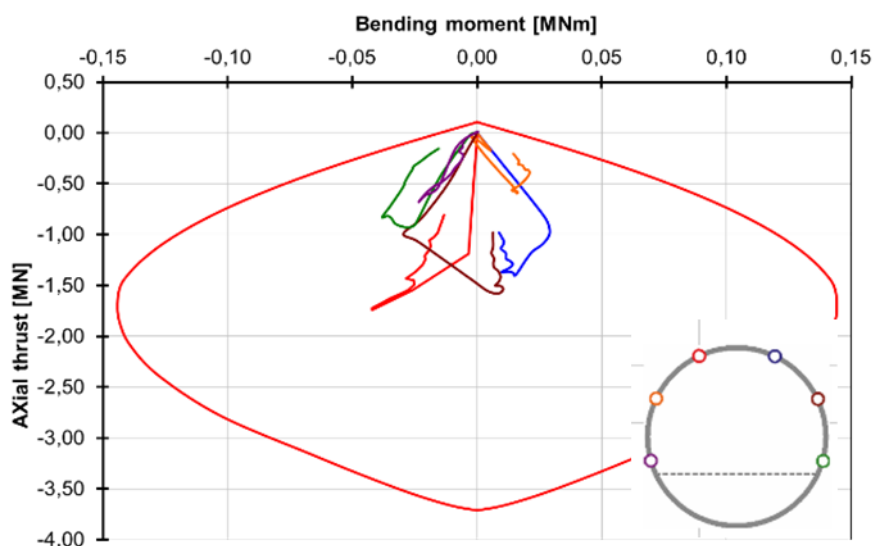
Figure 70. Piezometer evaluation – water pressure reduction in relation to a tunnel progress and versus time

### 5.5. Strain Gauges

Strains can be plotted versus time and/or construction progress. From pairs of strain measurements sectional forces in the lining can be derived, using the relationships of EC 2 [59]. The results have to be compared to the axial thrust – bending moment envelope determined for the respective shotcrete age (not for the nominal 28-days concrete strength) in order to obtain realistic utilisation estimates (Figure 71).



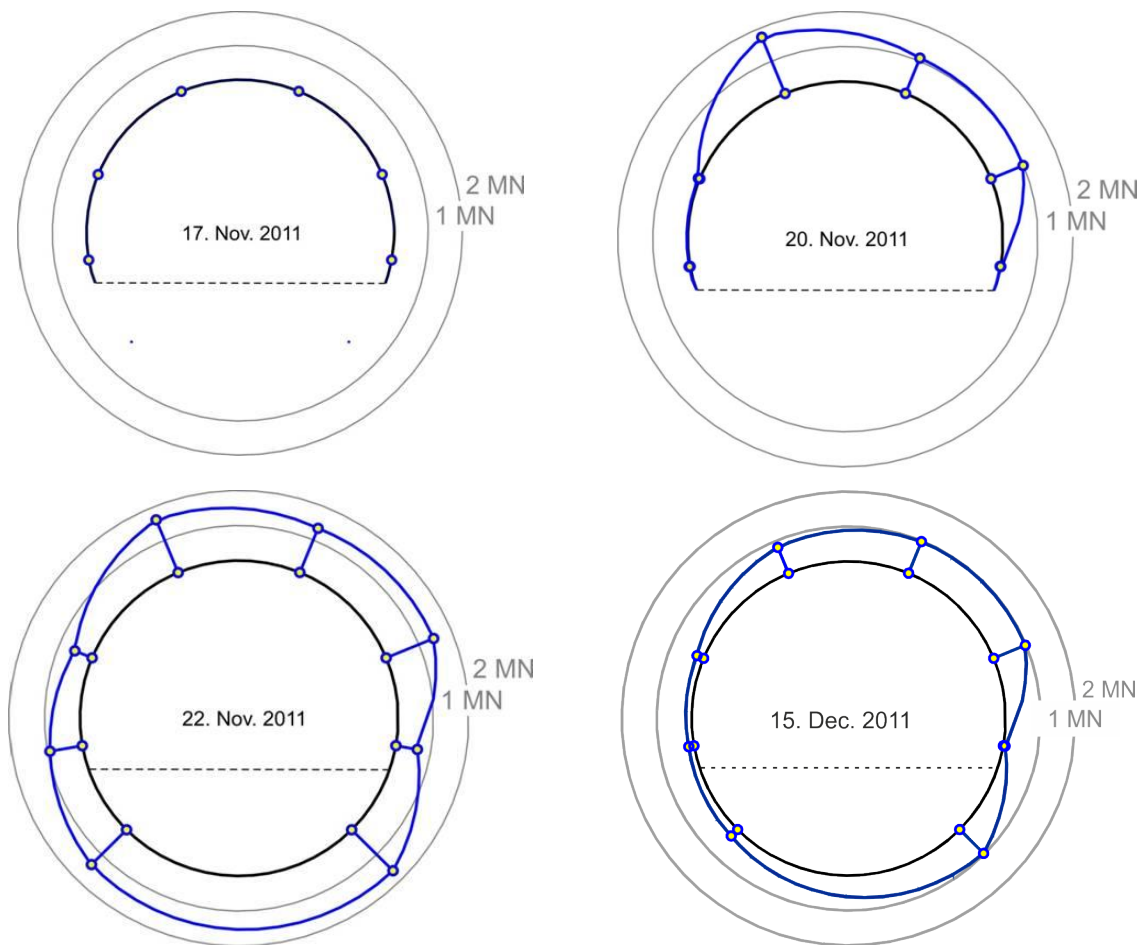
Section forces and capacity envelope, time = 2 days



Section forces and capacity envelope, time = 28 days

Figure 71. Development of thrust and bending moment for six strain gauge pairs installed in the top heading shotcrete lining during a monitoring time span of 28 days

The back calculated section forces should be interpolated between the respective strain gauge positions in the shotcrete shell, allowing the visualisation of their distribution in the lining and clear interpretation of the data. The results shown in Figure 72 clearly show that no axial thrust mobilisation is present near the base of the top heading. After the bench excavation and support are completed, the ring closure causes a stiffness increase of the entire support, leading to mobilisation of the axial thrust.



*Figure 72. Top left: the position of the strain gauges in the tunnel cross section and zero reading. Top right: lining thrust three days later – bench installation pending. Lower left: thrust distribution one day after bench installation. Lower right: thrust distribution at the final reading, indicating shotcrete relaxation*

## 5.6. Borehole Inclinometers

Readings along the instrument can be displayed versus time and/or construction progress. With inclinometers, ground movements outside the tunnel generally can be observed. An example of an inclinometer installed above the crown of a tunnel is shown together with the measured displacements (Figure 73). The purpose was recording displacements ahead of the advancing face.

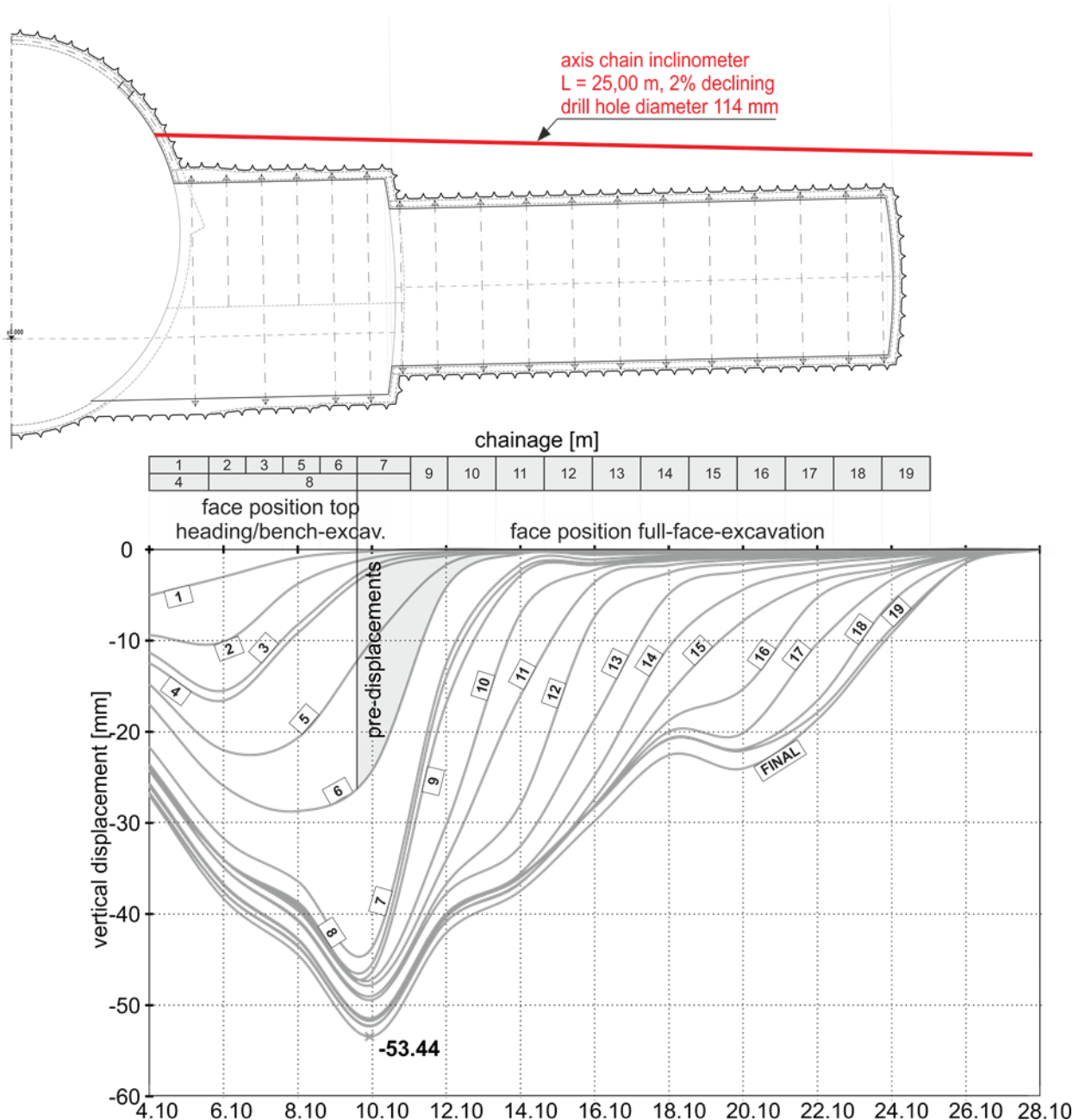


Figure 73. Example of nearly horizontal inclinometer installed above the tunnel crown, and measurement results [60]

Figure 74 shows a cumulative evaluation of vertical borehole inclinometer measurements in two directions under the assumption that the deepest point does not move. Alternatively, the incremental displacements of each measuring section can be displayed.

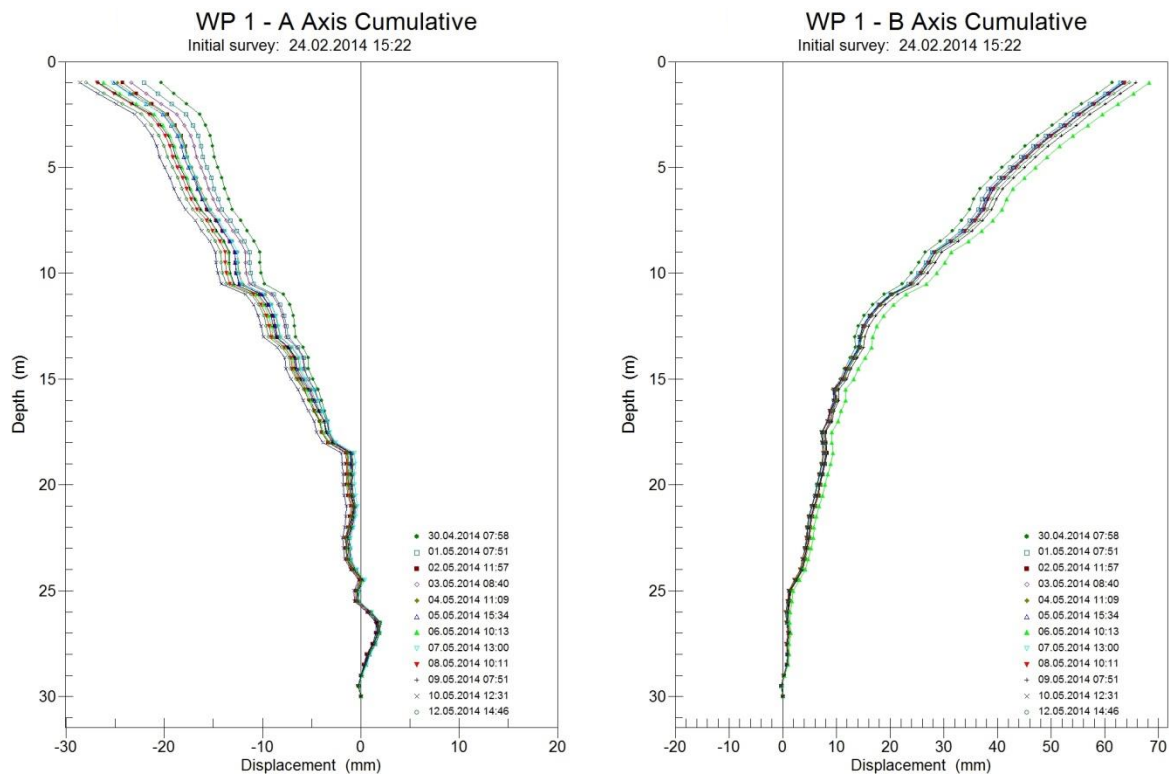


Figure 74. Example of cumulative evaluation of a vertical borehole inclinometer in two directions

The offsets in the lines at depths of approximately 11 m and 18 m indicate the development of shear bands. In addition pronounced creeping of the slope can be observed.

## 5.7. Invert Probe

As long as the probe can be moved through the entire length of the hose, the integrity of the invert can be interpreted.

## 5.8. Discontinuity location and orientation

### 5.8.1. Data acquired manually

Measured data are presented in hand sketches (Figure 75) and in stereographic projection (Figure 76).

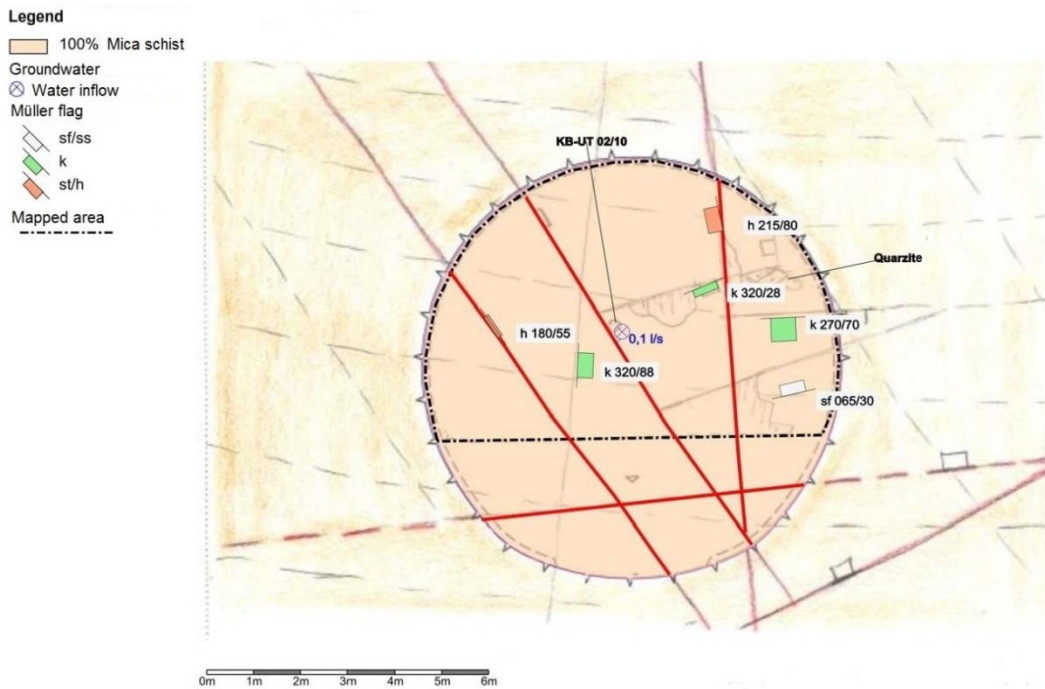


Figure 75. Sketch of face with discontinuity traces; discontinuity orientation shown as dip direction/dip, Müller flags indicate orientation; sf/ss: bedding, foliation; k: joint; st/h: fault, slickenside

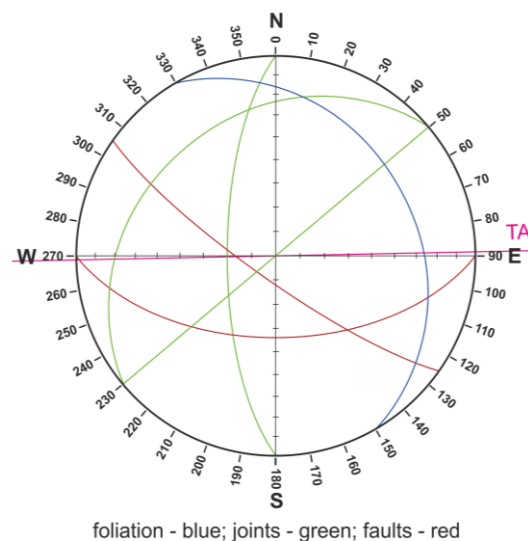


Figure 76. Representation of discontinuity orientations in stereographic projection (TA-tunnel axis orientation)

### 5.8.2. Data from contact free measurements

In the 3D images acquired using stereo photogrammetry or LiDAR, show the visible discontinuities, which are manually marked (areas or traces). The orientations are automatically displayed. The results are shown in (scaled) 3D images (Figure 77) and in stereographic projection (Figure 78).

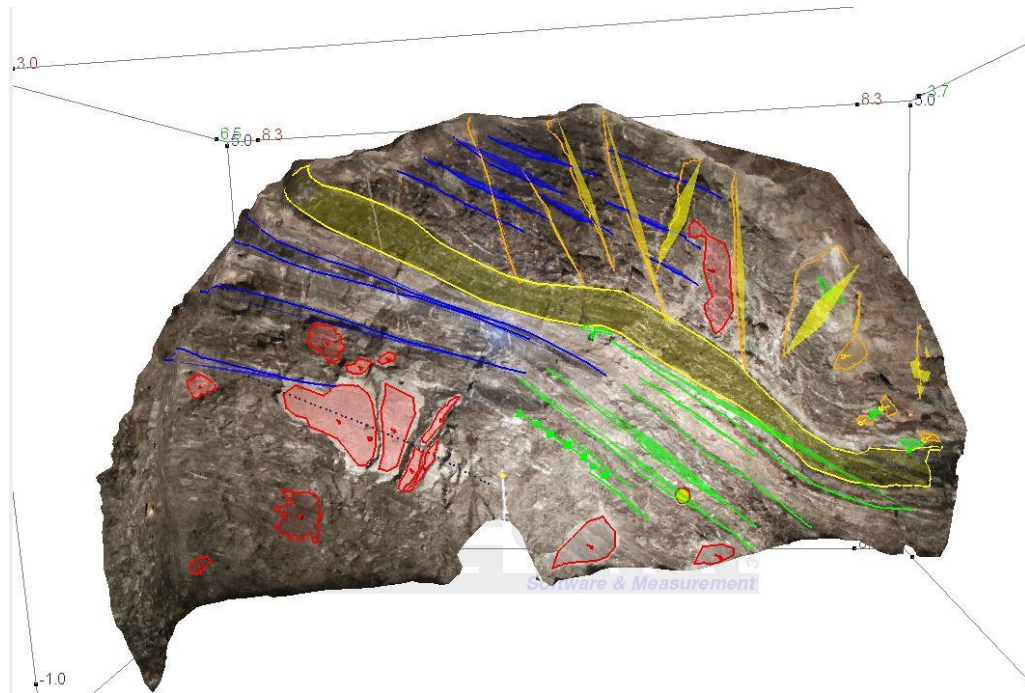


Figure 77. 3D image of a tunnel face with marked major rock structures (image 3GSM)

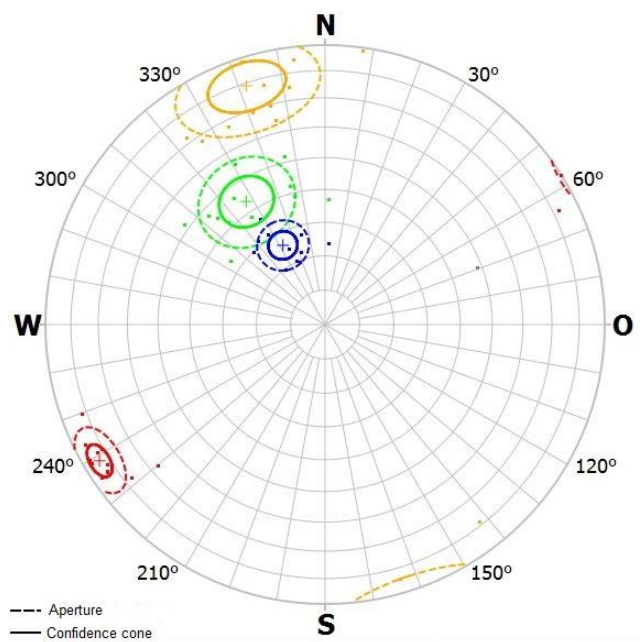


Figure 78. Hemispherical plot of orientation measurements

### 5.8.3. Analysis

The discontinuity data can be used to analyse the stability of blocks. Figure 79 shows removable blocks obtained from the discontinuity data. Using discontinuity properties and field stresses, factors of safety can be calculated.

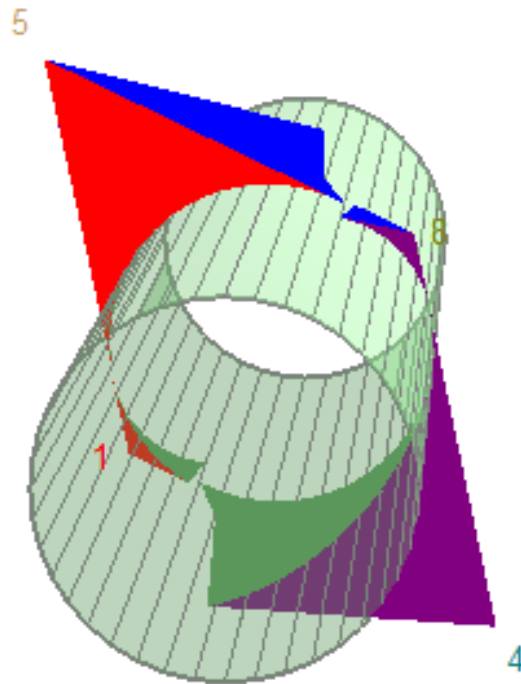


Figure 79. Example of assessment of block removability [61]

## **6. GEOTECHNICAL SAFETY MANAGEMENT PLAN**

Prior to construction, a geotechnical safety management plan (GSMP) shall be established. The contents and extent of the GSMP shall be adjusted to the specific requirements of a project. The goal is to achieve safe and economical construction, while effectively mitigating the residual risks [62, 63, 64].

### **6.1. Basics**

The following key issues shall be addressed in accordance with Eurocode EC 7 [65]:

- Design concept for determining excavation and support
- Determination of safety relevant parameters, including definition of expected behaviour and criteria for the assessment of the system stability on the basis of the expected ground conditions
- Monitoring concept, allowing a continuous comparison of expected and observed behaviour, including all organizational and technical requirements
- Management concept for cases where ground conditions and/or system behaviour deviate from that predicted, both for favourable and unfavourable deviations

During construction attention should be paid to following issues:

- Appropriate organisation of the site, and employment of qualified personnel
- Technical provisions for high quality observation of the system behaviour
- Efficient information flow and reporting

### **6.2. Contents**

#### **6.2.1. Contact details**

Contact details and functions of all persons relevant for the geotechnical safety management (GSM) shall be listed. For safety relevant functions it is advisable to nominate a proxy. In addition to the project team, this applies also for third parties (stakeholders), including owners or managers of buildings, structures (e.g. rails, roads) and utilities (e.g. gas or water pipe) in the area influenced by the site.

#### **6.2.2. Responsibilities**

The functions and responsibilities of all persons involved in the GSM shall be clearly defined. The GSMP shall contain stipulations regarding convening of meeting, intervals of meetings and participants.

#### **6.2.3. Geotechnical Meetings**

The purpose of geotechnical meetings is updating all stakeholders regarding:

- Observed behaviour in previous sections in comparison to the predicted
- Applied excavation and support in the previous sections
- Predicted ground conditions and system behaviour for the sections to be excavated
- Update geotechnical model, design and prediction if required

#### **6.2.4. Observation program**

In accordance with the project specific requirements a minimum observation program shall be specified with respect to observation method, observation intervals (distance of monitoring sections) and monitoring frequency according to Chapter 3.

#### **6.2.5. Definition of expected system behaviour (target behaviour)**

Prior to construction, the designer defines the target behaviour and effects of tunnel construction on the surface and structures, based on empirical or analytical stability considerations [1].

The safety relevant parameters related to the target behaviour and the limits and criteria of acceptable behaviour shall be determined on a project specific basis (see Section 3.3).

Typical parameters include:

- Tunnel displacements and their development during all construction phases
- Lining utilisation
- Face stability
- Water inflow
- Surface settlements, including spatial extent and development
- Displacements of buildings
- Allowable distortions or deflections of buildings and utilities (for example rails, water pipes and sewers, gas pipes, etc.)

The target behaviour shall be updated during construction, as required.

#### **6.2.6. Action levels**

The GSM generally consists of the following action levels (Figure 80):

- Observed behaviour complies with the target behaviour: continuous comparison to design values
- System behaviour on longer sections is more favourable than predicted: optimization of excavation and support
- System behaviour is less favourable than expected: immediate implementation of mitigation measures as specified in the GSMP, together with initiation of a crisis management

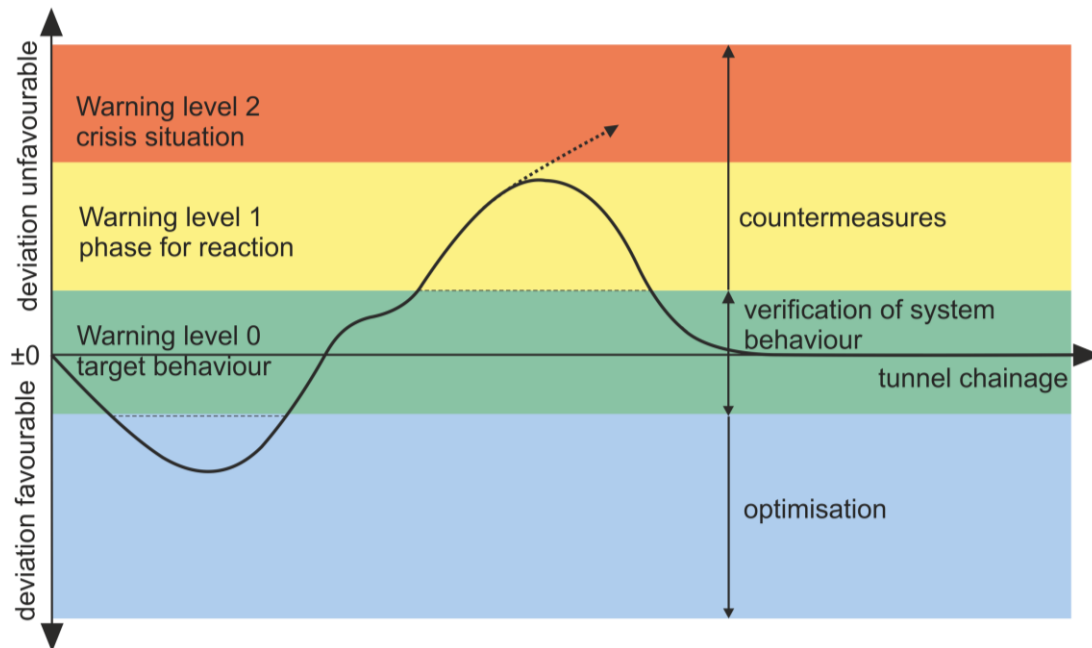


Figure 80. Warning and action levels [62, 64]

#### 6.2.7. Warning levels

The warning system shall be established to meet the project specific boundary conditions and requirements. As more experience is gained, the warning criteria should be updated during construction. The system consists of the warning levels listed in Table 6.

Table 6. Warning levels

Level		Characteristic
0		System behaviour within the range of target behaviour according to specifications of the design
1		Deviation from expected system behaviour; reaction phase
2	a	Imminent danger; risk is limited to the construction site; crisis situation
	b	Imminent danger; risk also affects third parties; crisis situation

Warning or alarm criteria can be fixed values, tendencies, or dangerous incidences, such as instability of tunnel or face, etc.

In order to obtain timely information regarding unfavourable developments, it can be advisable to define a pre-warning level.

In case of deviation from the target behaviour, the relevant warning level is declared.

The criteria for level 1 are characterized by a deviation from the target behaviour and/or high lining utilisation, but still providing sufficient safety against failure.

The criteria for level 2 describe repeated, progressive displacements, situations with only minimal safety against failure or including local failure (i.e. criteria at this level are already classified as alarm values or alarm criteria). If risks are limited to the construction site, level 2a is declared. In case third parties are affected, the measures according to level 2b are implemented.

The criteria for level 1 shall be set in such a way that mitigation measures can be handled by the means and methods available of site.

When reaching or exceeding the criteria of level 2, special procedures for the crisis management shall be implemented. The priority of action is as follows:

1. Ensure public safety
2. Ensure safety of persons involved in the project
3. Ensure stability of structure

The warning level can be revoked in case of:

- Stability of tunnel is proven by observation, respectively no influence on surface, utilities and structures (if applicable)
- Mitigation measures are implemented and their effectiveness has been proven

As an example in Table 7, typical criteria for values, tendencies and incidents as well as the corresponding mitigation measures are listed.

Table 7. Example of warning, alarm criteria and measures

	Criteria	Measures
<b>Level 0</b>  <b>Target behaviour</b>	<ul style="list-style-type: none"> <li>Behaviour of the structure as expected, no warning level reached</li> </ul>	<ul style="list-style-type: none"> <li>No measures</li> </ul>
<b>Level 1</b>  <b>Warning level</b>	<ul style="list-style-type: none"> <li>Reaching a warning level</li> <li>Atypical displacement behaviour of the tunnel or the surface, such as sudden displacement increases, increasing displacement trend, etc.</li> <li>Smaller unforeseen events in the tunnel, such as overbreak, larger water inflow, etc.</li> </ul>	<ul style="list-style-type: none"> <li>Immediate notification of all involved parties</li> <li>Implementation of predefined measures</li> <li>Special geotechnical meeting as required</li> <li>Increase of reading frequency</li> </ul>
<b>Level 2a</b>  <b>Alarm level without effects on third parties</b>	<ul style="list-style-type: none"> <li>Reaching an alarm level</li> <li>Repeated, progressive displacement increases with or without clearly allocable construction activity</li> <li>Indicators for stability problem of the tunnel (high probability of collapse)</li> <li>Water ingress in the tunnel</li> </ul>	<ul style="list-style-type: none"> <li>Implementation of crisis management plan</li> <li>Implementation of predefined measures</li> <li>Immediate measures to be taken by the contractor and the site supervision</li> <li>Immediate notification of all involved parties</li> <li>Call a geotechnical meeting in order to fix further measures</li> <li>Increase of reading frequency</li> </ul>
<b>Level 2b</b>  <b>Alarm level, with effects on third parties</b>	<ul style="list-style-type: none"> <li>As stage 2a, with effects on third parties</li> </ul>	<ul style="list-style-type: none"> <li>Implementation of crisis management</li> <li>Implementation of predefined measures</li> <li>Immediate measures to be taken by the contractor and the site supervision</li> <li>Immediate notification of all involved parties and responsible third parties</li> <li>Protection of the personnel and public (e.g. block access to hazard area, track lock, road block, evacuation of buildings, etc.)</li> <li>Call a geotechnical meeting in order to fix further measures</li> <li>Increase of reading frequency</li> </ul>

### **6.2.8. Organisation and communication**

The following organisational structure shall be provided at a tunnel site, independent of the contractual set-up.

#### **6.2.8.1. Information gathering**

All personnel having access to relevant information from monitoring data or observations in the tunnel shall be instructed that all such information shall be reported without delay to the data collection unit (typically the site supervision). Such personnel are typically: surveyors, geotechnical engineers, geologists, supervising personnel, miners, technical managers, etc.

#### **6.2.8.2. Data collection, communication centre and management of integrated process**

An organisational unit for following tasks shall be established:

- Collect and consolidate observations from the tunnel, geological interpretation, monitoring data, etc. are. Such unit may typically be the supervising Engineer
- Distribute all relevant information to the other units involved in the process, respectively stored in a data base, which is accessible by all stakeholders
- Manage the overall process of all involved parties

For efficient communication it is advisable to make joint site visits of representatives of the contractor, the site supervision, geologist and geotechnical engineer. A short routine meeting after the site visit allows exchange of information. If required, changes in the excavation and support can be determined directly at this meeting.

#### **6.2.8.3. Analysis**

One organisational unit shall be defined to analyse all observations in a broad context, including geological aspects, monitoring data, in-situ observations, etc. Such unit may typically be the geotechnical engineer (or team). A person, competent in geotechnical and structural engineering evaluates and interprets the monitoring data under consideration of additional information, such as geological or hydrogeological information or other relevant observations on site.

The geotechnical engineer submits the evaluated and interpreted data in the form of a summary report to the site supervision, the contractor, and the designer not later than the specified time. In case of unusual events, the geotechnical engineer immediately reports to the stakeholders.

The summary report usually is prepared as a table (see Table 8). The table allows an immediate overview regarding the warning levels of each measuring sections. In case of deviations from the normal behaviour, the results of the analysis and proposed measures are included in the report.

Table 8. Example of summary report in form of a table

Element		Measuring section	Warning and alarm level			Geotechnical explanation	Required measures
			0	1	2		
			normal	deviation	critical		
Lot A1	Underground	1181	√				
		1191	√				
		1202	√				
		1210		!		(1)	additional bolting
		1220		!		(1)	additional bolting
		1229	√			(2)	Additional measuring section at 1235
		Geotechnical Explanation:					
		(1) strong asymmetric displacements (left sidewall larger), .....					
		(2) increase in crown settlements, .....					

For complex projects with high risk potential, an independent geotechnical expert can be involved, who has access to all data and information, and performs an independent stability assessment. Depending on the specific conditions of the project, this can be a tunnelling or geotechnical expert, or checking engineer.

#### 6.2.8.4. Design of corrective measures

In case of deviation from the expected system behaviour, one organisational unit shall be responsible for designing corrective measures, or selecting such measures from a pre-designed catalogue. The organisational unit typically includes the geotechnical engineer, the designer, the client, and/or client's (supervising) Engineer and (especially in case of emergency) the contractor. This organisational unit shall provide a proposal to the decision unit.

### 6.2.8.5. Decision

It shall be made clear which unit or group of people is expected to make decisions on corrective measures. The organisation shall ensure that the flow of information between these units is well defined, and where necessary, complemented with time limits regarding dissemination of the information. The flow of information shown in Figure 81 has proven to be efficient. Project specific modifications may be required.

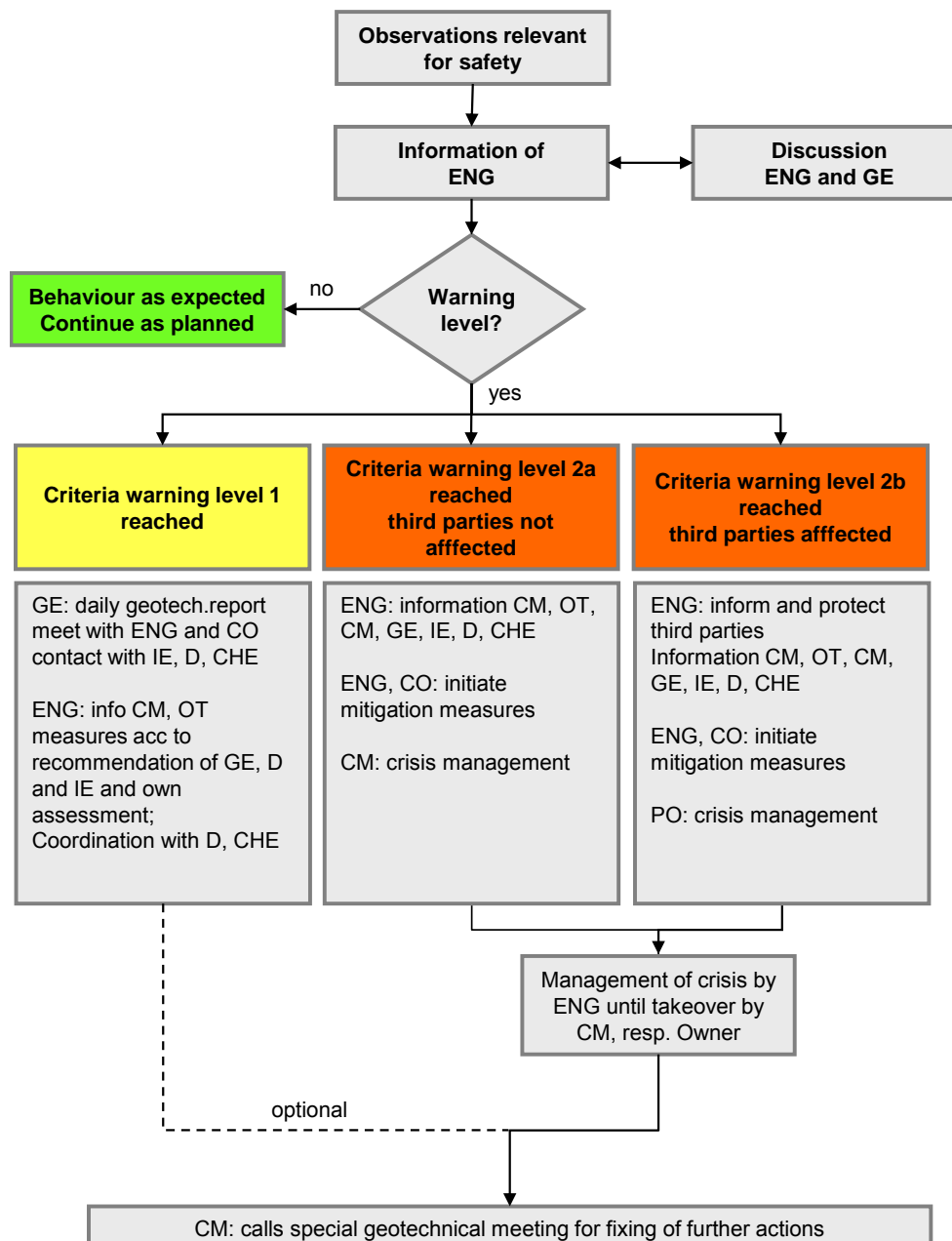


Figure 81. Example of procedure in Geotechnical Safety Management Plan; Abbreviations: GE - Geotechnical Engineer on site, ENG – Engineer (site supervision), CO – Contractor, IE – Independent Expert, D – Designer, CHE – Checking Engineer, CM - Construction Management, OT – Owners technical representative, PO – Owners Project Representative

#### **6.2.8.6. Leadership**

The functions may be different in case of routine work and in case of emergency. In the case of emergency it shall be clear who is in charge of coordinating all above functions.

### **6.3. Priorities**

In the geotechnical safety management plan the site-specific priorities shall be defined for different possible emergency scenarios.

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