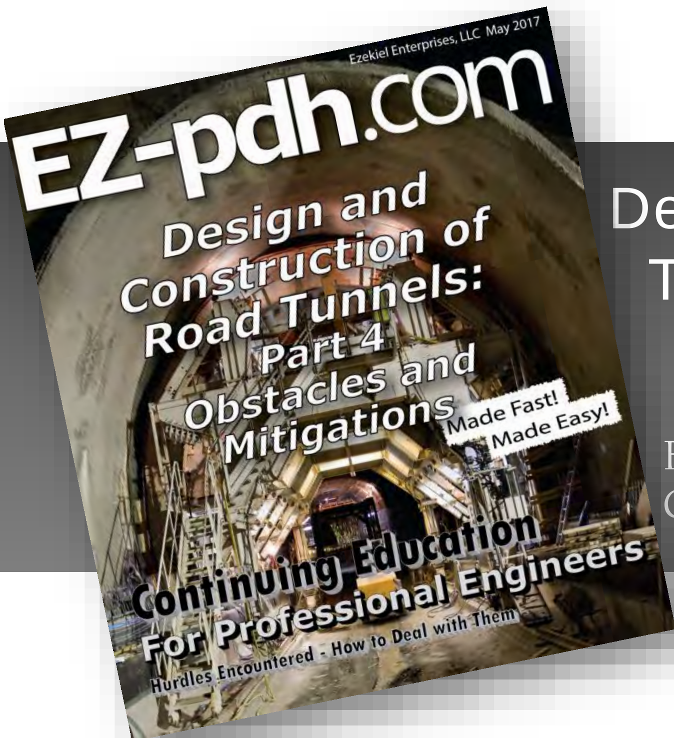

Design and Construction of Road Tunnels: Part 4 Obstacles and Mitigation

Five (5) Continuing Education Hours
Course #CV7054

Approved Continuing Education for Licensed Professional Engineers

EZ-pdh.com
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Course Description:

The Design and Construction of Road Tunnels: Part 4 Obstacles and Mitigation course satisfies five (5) hours of professional development.

The course is designed as a distance learning course that enables the practicing professional engineer to identify and handle many of the hurdles encountered during tunnel construction.

Objectives:

The primary objective of this course is enable the student to understand the reasons and methods to consider regarding seismic activity in the design process. The concepts of mined/bored tunnel construction engineering. Also, how to monitor the performance of the tunnel construction process, and how to identify, characterize and repair tunnels.

Grading:

Students must achieve a minimum score of 70% on the online quiz to pass this course. The quiz may be taken as many times as necessary to successful pass and complete the course.

A copy of the quiz questions are attached to last pages of this document.

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CHAPTER 13
SEISMIC CONSIDERATIONS

13.1 INTRODUCTION

Tunnels, in general, have performed better during earthquakes than have above ground structures such as bridges and buildings. Tunnel structures are constrained by the surrounding ground and, in general, can not be excited independent of the ground or be subject to strong vibratory amplification, such as the inertial response of a bridge structure during earthquakes. Another factor contributing to the reduced tunnel damage is that the amplitude of seismic ground motion tends to reduce with depth below the ground surface. Adequate design and construction of seismic resistant tunnel structures, however, should never be overlooked, as moderate to major damage has been experienced by many tunnels during earthquakes, as summarized by Dowding and Rozen (1978), Owen and Scholl (1981), Sharma and Judd (1991), and Power et al. (1998), among others. The greatest incidence of severe damage has been associated with large ground displacements due to ground failure, i.e., fault rupture through a tunnel, landsliding (especially at tunnel portals), and soil liquefaction. Ground shaking in the absence of ground failure has produced a lower incidence and degree of damage in general, but has resulted in moderate to major damage to some tunnels in recent earthquakes. The most recent reminder of seismic risk to underground structures under the ground shaking effect is the damage and near collapse at the Daikai and Nagata subway stations (Kobe Rapid Transit Railway) during the 1995 Kobe Earthquake in Japan. Near-surface rectangular cut-and-cover tunnels and immersed tube tunnels in soil have also been vulnerable to transient seismic lateral ground displacements, which tend to cause racking of a tunnel over its height and increased lateral pressures on the tunnel walls. Their seismic performance could be vital, particularly when they comprise important components of a critical transportation system (e.g., a transit system) to which little redundancy exists.

The general procedure for seismic design and analysis of tunnel structures should be based primarily on the ground deformation approach (as opposed to the inertial force approach); i.e., the structures should be designed to accommodate the deformations imposed by the ground. The analysis of the structure response can be conducted first by ignoring the stiffness of the structure, leading to a conservative estimate of the ground deformations. This simplified procedure is generally applicable for structures embedded in rock or very stiff/dense soil. In cases where the structure is stiff relative to the surrounding soil, the effect of soil-structure interaction must be taken into consideration. Other critical conditions that warrant special seismic considerations include cases where a tunnel intersects or meets another tunnel (e.g., tunnel junction or tunnel/cross-passage interface) or a different structure (such as a ventilation building). Under these special conditions, the tunnel structure may be restrained from moving at the junction point due to the stiffness of the adjoining structure, thereby inducing stress concentrations at the critical section. Complex numerical methods are generally required for cases such as these where the complex nature of the seismic soil-structure interaction system exists.

13.2 DETERMINATION OF SEISMIC ENVIRONMENT

13.2.1 Earthquake Fundamental

General: Earthquakes are produced by abrupt relative movements on fractures or fracture zones in the earth's crust. These fractures or fracture zones are termed *earthquake faults*. The mechanism of fault movement is elastic rebound from the sudden release of built-up strain energy in the crust. The built-up strain energy accumulates in the earth's crust through the relative movement of large, essentially intact pieces of the earth's crust called *tectonic plates*. This relief of strain energy, commonly called *fault rupture*, takes place along the *rupture zone*. When fault rupture occurs, the strained rock rebounds elastically. This rebound produces vibrations that pass through the earth crust and along the earth's surface, generating the ground motions that are the source of most damage attributable to earthquakes. If the fault along which the rupture occurs propagates upward to the ground surface and the surface is uncovered by sediments, the relative movement may manifest itself as *surface rupture*. Surface ruptures are also a source of earthquake damage to constructed facilities including tunnels.

The major tectonic plates of the earth's crust are shown in Figure 13-1 (modified from Park, 1983). There are also numerous smaller, minor plates not shown on this figure. Earthquakes also occur in the interior of the plates, although with a much lower frequency than at plate boundaries.

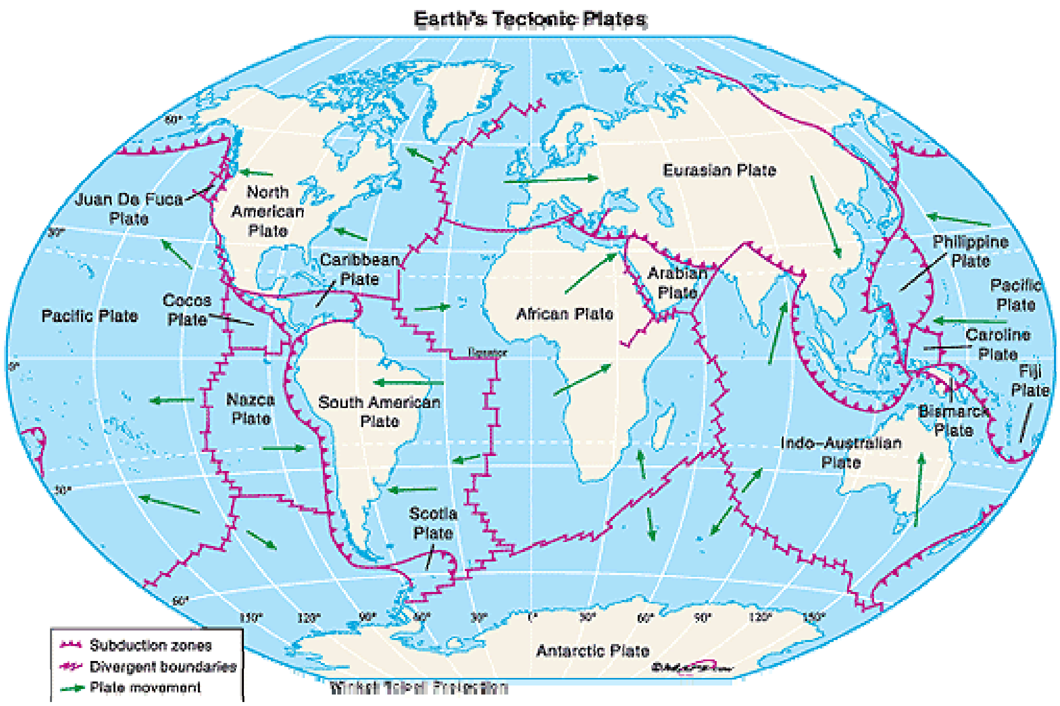


Figure 13-1 Major Tectonic Plates and Their Approximate Direction of Movement. (Source: www.maps.com)

For the continental United States, the principal tectonic plate boundary is along the western coast of the continent, where the North American Plate and the Pacific Plate are in contact. In California, the boundary between these plates is a transform fault wherein the relative movement is generally one of lateral slippage of one plate past the other. Elsewhere along the west coast (e.g., off the coast of Oregon, Washington, and Alaska), the plate boundary is a *subduction zone* wherein one plate dives (subducts) beneath the other plate. In the western interior of the United States, adjacent to the western edge of the American Plate, there may be subplates that have formed as a result of subcrustal flow. Earthquake sources in Utah and Montana may be attributable to such subplate sources. Earthquake source areas in the central and eastern United States and along the Saint Lawrence Valley are within the American Plate and are considered to be intraplate source zones. The mechanisms generating earthquakes in these intraplate zones are poorly understood, but may be related to relief of locked-in stresses from ancient tectonic movements, crustal rebound from the ice ages, re-adjustment of stress in the interior of the plate due to boundary loads, sediment load such as the Mississippi River basin, or other unrecognized mechanisms. Earthquakes in Hawaii are believed to be associated with an isolated plume of molten rock from the mantle referred to as a hot spot.

The intensity and impact of earthquakes may be as great or greater in the plate interiors as they are at the active plate boundaries. The differences between plate boundary and intraplate earthquakes is in their geographic spread and the frequency of occurrence. Earthquake activity is much greater along the plate boundaries than in the plate interior. However, ground motions from intraplate earthquakes tend to attenuate, or dissipate, much more slowly than those from plate boundary events. Plate boundary faults are relatively longer than those in the plate interior and tend to be associated with a smaller *stress drop* (the stress drop is the sudden reduction of stress across the fault plane during rupture), longer duration of shaking, and a more frequent rate of earthquake occurrence.

<u>Fault Movements:</u> Faults are created when the stresses within geologic materials exceed the ability of those materials to withstand the stresses. Most faults that exist today are the result of tectonic activity that occurred in earlier geological times. These faults are usually non-seismogenic (i.e. incapable of generating earthquakes, or inactive). However, faults related to past tectonism may be reactivated by present-day tectonism in seismically active areas and can also be activated by anthropogenic (man-made) activities such as impoundment of a reservoir by a dam or injection of fluids (e.g. waste liquids) deep into the subsurface. The maximum size of an earthquake on an anthropogenically reactivated fault is a subject of some controversy, but earthquakes as large as moment magnitude 6.5 have been attributed to reservoir impoundment.

Not all faults along which relative movement is occurring are a source of earthquakes. Some faults may be surfaces along which relative movement is occurring at a slow, relatively continuous rate, with an insufficient stress drop to cause an earthquake. Such movement is called *fault creep*. Fault creep may occur along a shallow fault, where the low overburden stress on the fault results in a relatively low threshold stress for initiating displacement along the fault. Alternatively, a creeping fault may be at depth in soft and/or ductile materials that deform plastically. Also, there may be a lack of frictional resistance or asperities (non-uniformities) along the fault plane, allowing steady creep and the associated release of the strain energy along the fault. Fault creep may also prevail where phenomena such as magma intrusion or growing salt domes activate small shallow faults in soft sediments. Faults generated by extraction of fluids (e.g., oil or water in southern California), which causes ground settlement and thus activates faults near the surface may also result in fault creep. Faults activated by other non-tectonic mechanisms, e.g. faults generated by gravity slides that take place in thick, unconsolidated sediments, could also produce fault creep.

Active faults that extend into crystalline bedrock are generally capable of building up the strain energy needed to produce, upon rupture, earthquakes strong enough to affect transportation facilities. Fault ruptures may propagate from the crystalline bedrock to the ground surface and produce ground rupture. Fault ruptures which propagate to the surface in a relatively narrow zone of deformation that can be traced back to the causative fault in crystalline rock are sometimes referred to as primary fault ruptures. Fault ruptures may also propagate to the surface in diffuse, distributed zones of deformation which cannot be traced directly back to the basement rock. In this case, the surface deformation may be referred to as secondary fault rupture.

Whether or not a fault has the potential to produce earthquakes is usually judged by the recency of previous fault movements. If a fault has propagated to the ground surface, evidence of faulting is usually found in geomorphic features associated with fault rupture (e.g., relative displacement of geologically young sediments). For faults that do not propagate to the ground surface, geomorphic evidence of previous earthquakes may be more subdued and more difficult to evaluate (e.g., near surface folding in sediments or evidence of liquefaction or slumping generated by the earthquakes). If a fault has undergone relative displacement in relatively recent geologic time (within the time frame of the current tectonic setting), it is reasonable to assume that this fault has the potential to move again. If the fault moved in the distant geologic past, during the time of a different tectonic stress regime, and if the fault has not moved in recent (Holocene) time (generally the past 11,000 years), it may be considered inactive. For some very important and critical facilities, such as those whose design is governed by the US Nuclear Regulatory Commission (NRC), a timeframe much longer than the 11,000-yr criterion has been used. In accordance with the US NRC regulations a fault is defined as “capable” (as opposed to “active”) if it has shown activity within the past 35,000 years or longer.

Geomorphic evidence of fault movement cannot always be dated. In practice, if a fault displaces the base of unconsolidated alluvium, glacial deposits, or surficial soils, then the fault is likely to be active. Also, if there is micro-seismic activity associated with the fault, the fault may be judged as active and capable of generating earthquakes. Microearthquakes occurring within basement rocks at depths of 7 to 20 km may be indicative of the potential for large earthquakes. Microearthquakes occurring at depths of 1 to 3 km are not necessarily indicative of the potential for large, damaging earthquake events. In the absence of geomorphic, tectonic, or historical evidence of large damaging earthquakes, shallow microtremors may simply indicate a potential for small or moderate seismic events. Shallow microearthquakes of magnitude 3 or less may also sometimes be associated with mining or other non-seismogenic mechanisms. If there is no geomorphic evidence of recent seismic activity and there is no microseismic activity in the area, then the fault may be inactive and not capable of generating earthquakes.

In some instances, fault rupture may be confined to the subsurface with no relative displacement at the ground surface due to the fault movement. Subsurface faulting without primary fault rupture at the ground surface is characteristic of almost all but the largest magnitude earthquakes in the central and eastern United States. Due to the rarity of large magnitude intraplate events, geological processes may erase surface manifestations of major earthquakes in these areas. Therefore, intraplate seismic source zones often must be evaluated using instrumental seismicity and paleoseismicity studies. This is particularly true if the intraplate sources are covered by a thick mantle of sediments, as in the New Madrid, Tennessee, and Charleston, South Carolina, intraplate seismic zones. Instrumental recording of small magnitude events can be particularly effective in defining seismic source zones.

Essentially all of the active faults with surface fault traces in the United States are shallow crustal faults west of the Rocky Mountains. However, not all shallow crustal faults west of the Rocky Mountains have surface fault traces. Several recent significant earthquakes along the Pacific Coast plate boundary (e.g., the 1987 Whittier Narrows earthquake and the 1994 Northridge earthquake) were due to rupture of thrust (compressional) faults that did not break the ground surface, termed *blind thrust* faults.

A long fault, like the San Andreas Fault in California or the Wasatch Fault in Utah, typically will not move along its entire length at any one time. Such faults typically move in portions, one segment at a time. An immobile (or "locked") segment, a segment which has remained stationary while the adjacent segments of the fault have moved, is a strong candidate for the next episode of movement.

<u>Type of Faults:</u> Faults may be broadly classified according to their mode, or style of relative movement. The principal modes of relative displacement are illustrated in Figure 13-2 and are described subsequently.

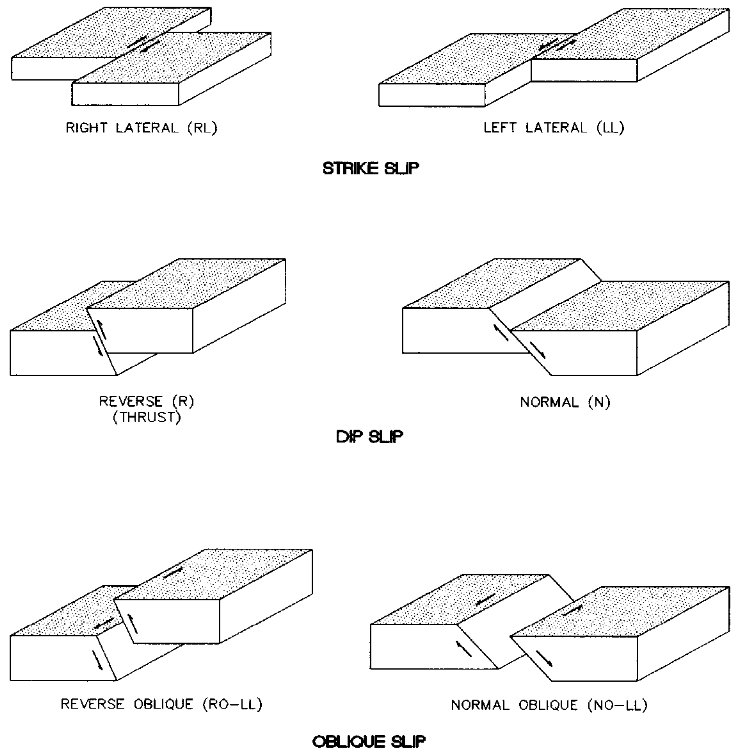


Figure 13-2 Types of Fault Movement

Strike Slip Faults: Faults along which relative movement is essentially horizontal (i.e., the opposite sides of the fault slide past each other laterally), are called strike slip faults. Strike slip faults are often essentially linear (or planar) features. Strike slip faults that are not fairly linear may produce complex surface features. The San Andreas fault is a strike slip fault that is essentially a north-south linear feature over most of its length. Strike slip faults may sometimes be aligned in en-echelon fashion wherein individual sub-parallel segments are aligned along a linear trend. En-echelon strike slip faulting is sometimes accompanied by step over zones where fault displacement is transferred from adjacent strike slip faults. Ground rupture patterns within these zones may be particularly complex.

Dip Slip Faults: Faults in which the deformation is perpendicular to the fault plane may occur due to either *normal* (extensional) or *reverse* (compressional) motion. These faults are referred to as *dip slip* faults. Reverse faults are also referred to as *thrust faults*. Dip slip faults may produce multiple fractures within rather wide and irregular fault zones.

Other Special Cases: Faults that show both strike slip and dip slip displacement may be referred to as *oblique slip faults*.

Earthquake Magnitude: *Earthquake magnitude*, M, is a measure of the energy released by an earthquake. A variety of different earthquake magnitude scales exist. The differences among these scales is attributable to the earthquake characteristic used to quantify the energy content. Characteristics used to quantify earthquake energy content include the local intensity of ground motions, the body waves generated by the earthquake, and the surface waves generated by the earthquake. In the eastern United States, earthquake magnitude is commonly measured as a (short period) *body wave magnitude*, m_b. However, the (long period) body wave magnitude, m_B, scale is also sometimes used in the central and eastern United States. In California, earthquake magnitude is often measured as a *local (Richter) magnitude*, M_L, or *surface wave magnitude*, M_s. The *Japan Meteorological Agency Magnitude* (M_{JMA}) scale is commonly used in Japan.

Due to limitations in the ability of some recording instruments to measure values above a certain amplitude, some of these magnitude scales tend to reach an asymptotic upper limit. To correct this, the *moment magnitude*, M_w, scale was developed by seismologists (Hanks and Kanamori, 1979). The moment magnitude of an earthquake is a measure of the kinetic energy released by the earthquake. M_w is proportional to the *seismic moment*, defined as a product of the material rigidity, fault rupture area, and the average dislocation of the rupture surface. Moment magnitude has been proposed as a unifying, consistent magnitude measure of earthquake energy content. Figure 13-3 (Heaton, *et al.*, 1986) provides a comparison of the various other magnitude scales with the moment magnitude scale.

<u>Hypocenter and Epicenter and Site-to-Source Distance:</u> The *hypocenter* (focus) of an earthquake is the point from which the seismic waves first emanate. Conceptually, it may be considered as the point on a fault plane where the slip responsible for an earthquake was initiated. The *epicenter* is a point on the ground surface directly above the hypocenter. Figure 13-4 shows the relationship between the hypocenter, epicenter, fault plane, and rupture zone of an earthquake. Figure 13-4 also shows the definition of the *strike* and *dip angles* of the fault plane.

The horizontal distance between the site of interest to the epicenter is termed epicentral distance, R_E, and is commonly used in the eastern United States. The distance between the site and the hypocenter (more widely used in the western United States) is termed hypocentral distance, R_H.

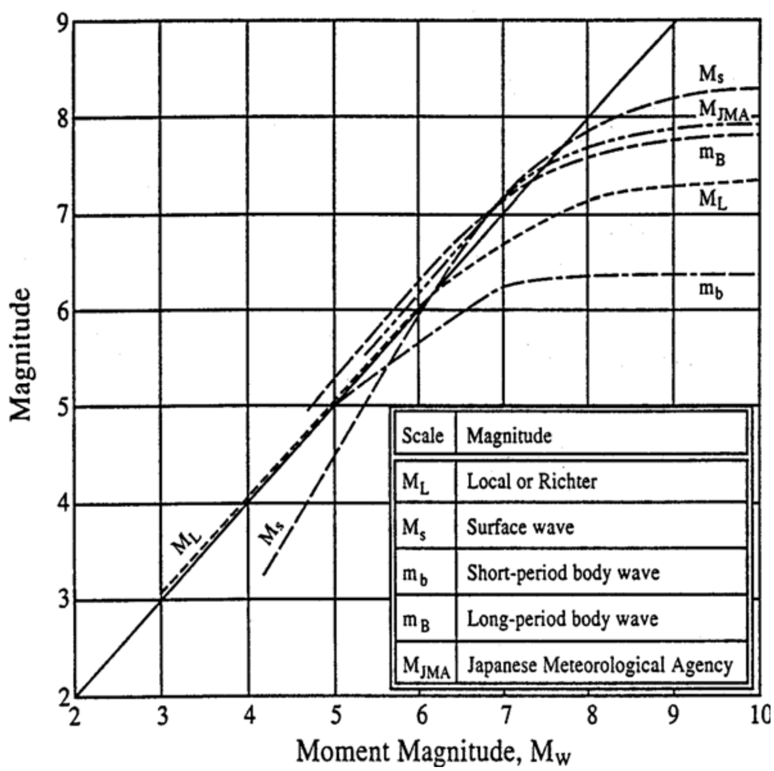


Figure 13-3 Comparison of Earthquake Magnitude Scales (Heaton, *et al.*, 1986)

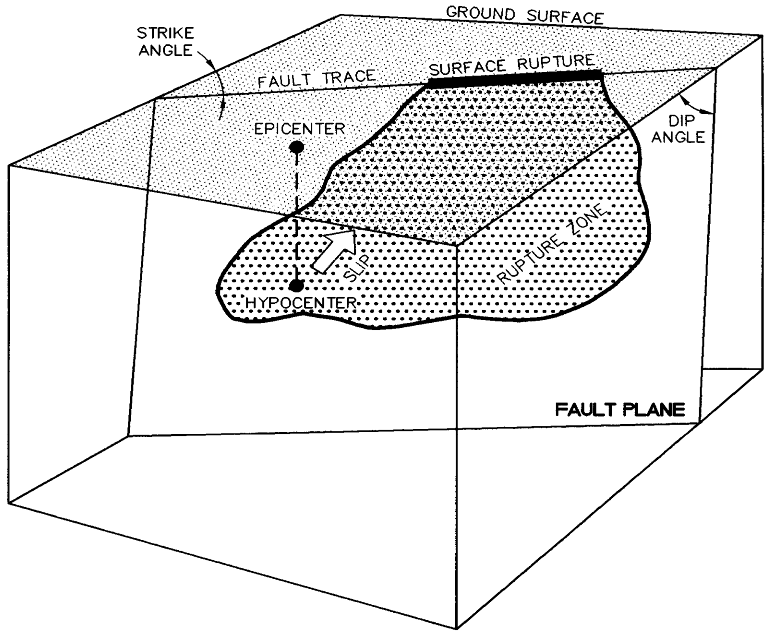


Figure 13-4 Definition of Basic Fault Geometry Including Hypocenter and Epicenter

13.2.2 Ground Motion Hazard Analysis

For the seismic design of underground tunnel facilities, one of the main tasks is to define the design earthquake(s) and the corresponding ground motion levels and other associated seismic hazards. The process by which design ground motion parameters are established for a seismic analysis is termed the *seismic hazard analysis*. Seismic hazard analyses generally involve the following steps:

- Identification of the seismic sources capable of strong ground motions at the project site
- Evaluation of the seismic potential for each capable source
- Evaluation of the intensity of the design ground motions at the project site

Identification of seismic sources includes establishing the type of fault and its geographic location, depth, size, and orientation. Seismic source identification may also include specification of a random seismic source to accommodate earthquakes not associated with any known fault. Evaluation of the seismic potential of an identified source involves evaluation of the earthquake magnitude (or range of magnitudes) that the source can generate and, often times, the expected rate of occurrence of events of these magnitudes.

Identification of capable seismic sources together with evaluation of the seismic potential of each capable source may be referred to as *seismic source characterization*. Once the seismic sources are characterized, the intensity of ground motions at the project site from these sources must be characterized. There are three general ways by which the intensity of ground motions at a project site is assessed in practice. They are, in order of complexity: (1) use of existing hazard analysis results published by credible agencies such as US Geological Survey (USGS) and some State agencies; (2) project-specific and site-specific deterministic seismic hazard evaluation; and (3) project-specific and site-specific probabilistic seismic hazard evaluation. Which particular approach is adopted may depend on the importance and complexity of the project and may be dictated by regulatory agencies.

The choice of the design ground motion level, whether based upon probabilistic or deterministic analysis, cannot be considered separately from the level of performance specified for the design event. Sometimes, facilities may be designed for multiple performance levels, with a different ground motion level assigned to each performance level, a practice referred to as performance based design. Common performance levels used in design of transportation facilities include protection of life safety and maintenance of function after the event. A safety level design earthquake criterion is routinely employed in seismic design. Keeping a facility functional after a large earthquake adds another requirement to that of simply maintaining life safety, and is typically required for critical facilities.

The collapse of a modern transportation tunnel (particularly for mass transit purpose) during or after a major seismic event could have catastrophic effects as well as profound social and economical impacts. It is typical therefore for modern and critical transportation tunnels to be designed to withstand seismic ground motions with a return period of 2,500 years, (corresponding to 2 % probability of exceedance in 50 years, or 3% probability of exceedance in 75 years). In addition, to avoid lengthy down time and to minimize costly repairs, a modern and critical transportation tunnel is often required to withstand a more frequent earthquake (i.e., a lower level earthquake) with minimal damage. The tunnel should be capable of being put immediately back in service after inspection following this lower level design earthquake. In the high seismic areas, this lower level earthquake is generally defined to have a 50% probability of probability of exceedance 75 years, corresponding to a 108-year return period. In the eastern United States, where earthquake occurrence is much less frequent, the lower level design earthquake for modern and critical transportation tunnels is generally defined at a higher return period such as 500 years.

<u>Use Of Existing Hazard Analysis Results</u>: Information used for seismic source characterization can often be obtained from publications of the United States Geological Survey (USGS), or various state agencies. These published results are often used because they provide credibility for the designer and may give the engineer a feeling of security. However, if there is significant lag time between development and publication, the published hazard results may not incorporate recent developments on local or regional seismicity. Furthermore, there are situations where published hazard results may be inadequate and require site-specific seismic hazard evaluation. These situations may include: (1) the design earthquake levels (e.g., in terms of return period) are different than those assumed in the published results, (2) for sites located within 6 miles of an active surface or shallow fault where near-field effect is considered important, and (3) the published hazard results fail to incorporate recent major developments on local or regional seismicity.

Seismic hazard maps that include spectral acceleration values at various spectral periods have been developed by USGS under the National Earthquake Hazard Reduction Program (NEHRP). Map values for peak and spectral accelerations with a probability of being exceeded of 2 percent, 5 percent, and 10 percent in 50 years (corresponding approximately to 2,500-yr, 1,000-yr, and 500-yr return period, respectively) can be recovered in tabular form. Figure 13-5 below shows an example of the national ground motion hazard maps in terms of peak ground acceleration (in Site Class B – Soft Rock Site) for an event of 2% probability of exceedance in 50 Years (i.e., 2,500-yr Return Period). In addition, USGS also provides information (e.g., the de-aggregated hazard) that can be used to estimate the representative "magnitude and distance" for a site in the continental United States.

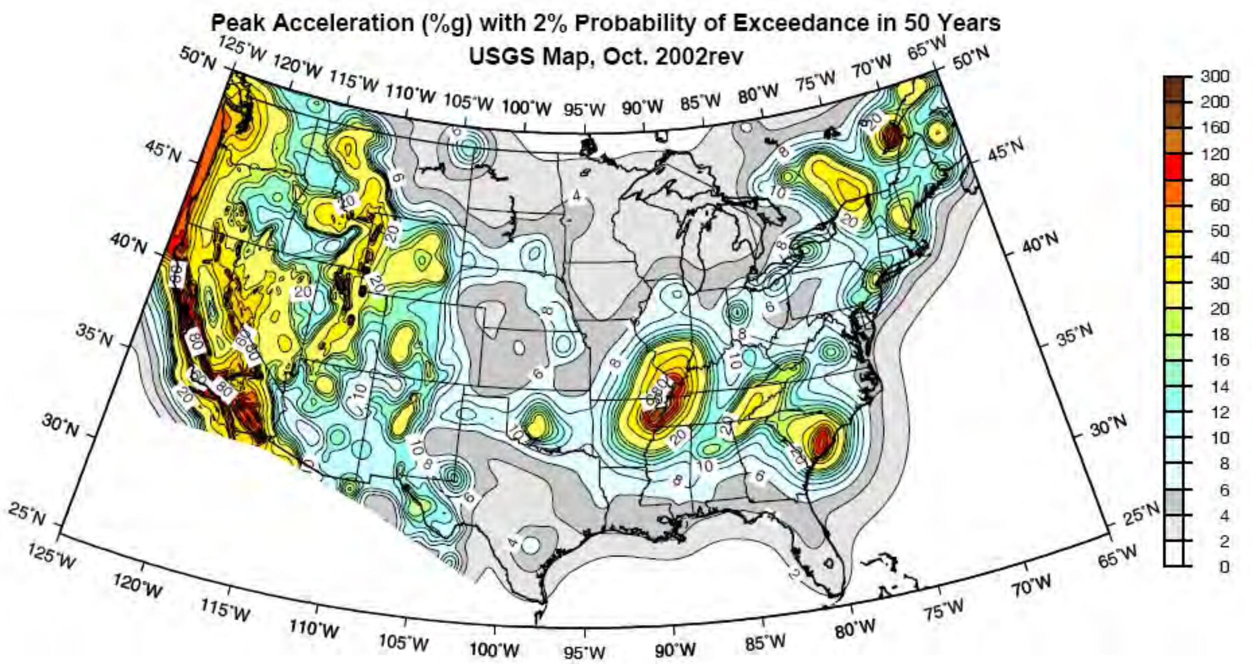


Figure 13-5 National Ground Motion Hazard Map by USGS (2002) - Peak Ground Acceleration with 2% Probability of Exceedance in 50 Years (2,500-yr Return Period) - for Site Class B, Soft Rock

<u>The Deterministic Hazard Analysis Approach:</u> In a deterministic seismic hazard analysis, the seismologist performing the analysis first identifies the capable seismic sources and assigns a maximum magnitude to each source. Then, the intensity of shaking at the site from each capable source is calculated and the design earthquake is identified based on the source capable of causing the greatest damage. The steps in a deterministic seismic hazard analysis are as follows:

1. Establish the location and characteristics (e.g., style of faulting) of all potential earthquake sources that might affect the site. For each source, assign a representative earthquake magnitude.
2. Select an appropriate attenuation relationship and estimate the ground motion parameters at the site from each capable fault as a function of earthquake magnitude, fault mechanism, site-to-source distance, and site conditions. Attenuation relationships discriminate between different styles of faulting and between rock and soil sites.
3. Screen the capable (active) faults on the basis of magnitude and the intensity of the ground motions at the site to determine the governing source.

The deterministic analysis approach provides a framework for the evaluation of worst-case scenarios at a site. It provides little information about the likelihood or frequency of occurrence of the governing earthquake. If such information is required, a probabilistic analysis approach should be used to better define the seismic ground motion hazard.

<u>The Probabilistic Hazard Analysis Approach:</u> A probabilistic seismic hazard analysis incorporates the likelihood of a fault rupturing and the distribution of earthquake magnitudes associated with fault rupture into the assessment of the intensity of the design ground motion at a site. The objective of a probabilistic seismic hazard analysis is to compute, for a given exposure time, the probability of exceedance

corresponding to various levels of a ground motion parameter (e.g., the probability of exceeding a peak ground acceleration of 0.2 g in a 100-year period). The ground motion parameter may be either a peak value (e.g., peak ground acceleration) or a response spectra ordinate associated with the strong ground motion at the site. The probabilistic value of the design parameter incorporates both the uncertainty of the attenuation of strong ground motions and the randomness of earthquake occurrences. A probabilistic seismic hazard analysis usually includes the following steps, as illustrated in Figure 13-6:

1. Identify the seismic sources capable of generating strong ground motion at the project site. In areas where no active faults can be readily identified it may be necessary to rely on a purely statistical analysis of historical earthquakes in the region.
2. Determine the minimum and maximum magnitude of earthquake associated with each source and assign a frequency distribution of earthquake occurrence to the established range of magnitudes. The Gutenberg-Richter magnitude-recurrence relationship (Gutenberg and Richter, 1942) is the relationship used most commonly to describe the frequency distribution of earthquake occurrence. While the maximum magnitude is a physical parameter related to the fault dimensions, the minimum magnitude may be related to both the physical properties of the fault and the constraints of the numerical analysis.
3. For each source, assign an attenuation relationship on the basis of the style of faulting. Uncertainty is usually assigned to the attenuation relationships based upon statistical analysis of attenuation in previous earthquakes.
4. Calculate the probability of exceedance of the specified ground motion parameter for a specified time interval by integrating the attenuation relationship over the magnitude distribution for each source and summing up the results.

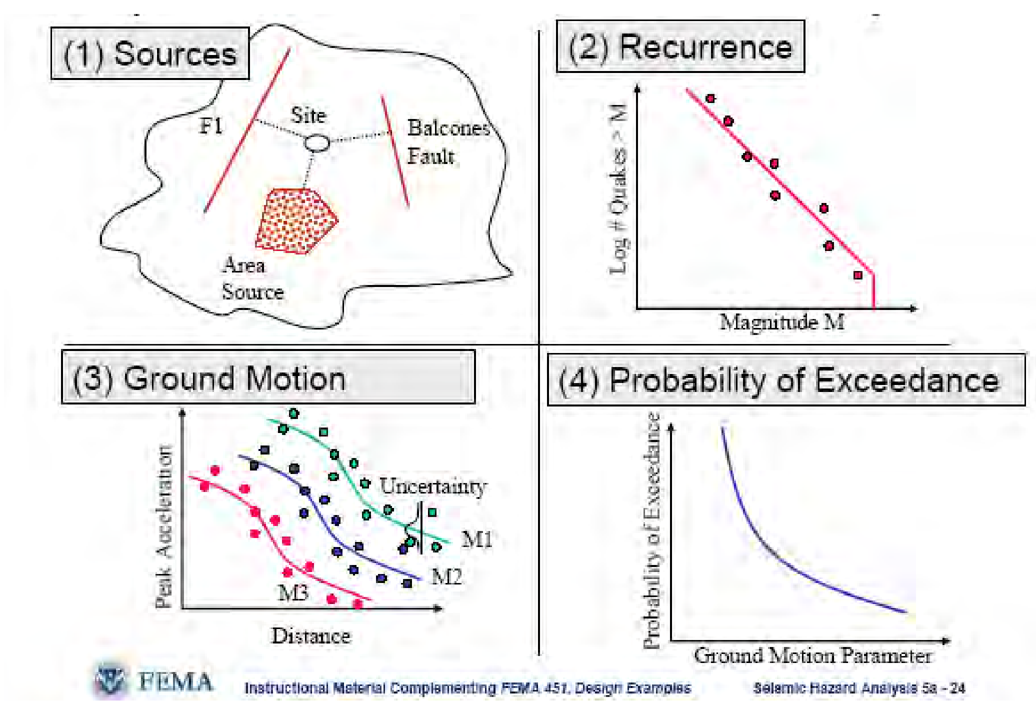


Figure 13-6 General Procedure for Probabilistic Seismic Hazard Analysis

13.2.3 Ground Motion Parameters

Once the design earthquake events are defined, design ground motion parameters are required to characterize the design earthquake events. Various types of ground motion parameters may be required depending on the type of analysis method used in the design. In general, ground motions can be characterized by three translational components (e.g., longitudinal, transverse, and vertical with respect to the tunnel axis). The various types of common ground motion parameters are described in the following paragraphs.

Peak Ground Motion Parameters: Peak ground acceleration (PGA), particularly in the horizontal direction, is the most common index of the intensity of strong ground motion at a site. Peak ground velocity (PGV) and peak ground displacement (PGD) are also used in some engineering analyses to characterize the damage potential of ground motions. For seismic design and analysis of underground structures including tunnels, the PGV is as important as the PGA because ground strains (or the differential displacement between two points in the ground) can be estimated using the PGV. PGA values are generally available from published hazard results such as those from the USGS hazard study. Attenuation relations are also generally available for estimating PGA values. However, there has been little information in the past for estimating the PGV values. Previous studies have attempted to correlate the PGV with PGA by establishing PGV-to-PGA ratios (as a function of earthquake magnitudes, site soil conditions, and source-to-site distance in some cases). However, these correlations were derived primarily from ground motion database in the Western United States (WUS) and failed to account for the different ground motion characteristics in the Central and Eastern United States (CEUS). Recent study (NCHRP-12-70, 2008) has found that PGV is strongly correlated with the spectral acceleration at 1.0 second (S_1). Using published strong motion data, regression analysis was conducted and the following correlation has been recommended for design purposes.

$$PGV = 0.394 \times 10^{0.434C} \qquad 13\text{-}1$$

Where:

PGV is in in/sec

$$C = 4.82 + 2.16 \log_{10} S_1 + 0.013 \left[2.30 \log_{10} S_1 + 2.93\right]^2 \qquad 13\text{-}2$$

The development of the PGV-S_1 correlation is based on an extensive earthquake database established from recorded accelerograms representative of both rock and soil sites for the WUS and CEUS. The earthquake magnitude was found to play only a small role and is not included in the correlation in developing Equations 13-1 and 13-2. Equation 13-1 is based on the mean plus one standard deviation from the regression analysis (i.e., 1.46 x the median value) for conservatism.

Design Response Spectra: Response spectra represent the response of a damped single degree of freedom system to ground motion. Design response spectra including the consideration of soil site effects can be established using code-specified procedures such as those specified in the NEHRP (National Earthquake Hazards Reduction Program) publications or the new AASHTO LRFD Guide Specifications using the appropriate design earthquake parameters consistent with the desirable design earthquake hazard levels (refer to discussions in Section 13.2.2). Figure 13-7 illustrates schematically the construction of design response spectra using the NEHRP procedure. The terms and parameters used in Figure 13-7 are documented in details in NEHRP 12-70 (2008) and in AASHTO LRFD Bridge Design Specifications (2008 Interim Provisions). Alternatively, project-specific and site-specific hazard analysis can also be performed to derive the design response spectra. Site-specific dynamic soil response analysis can also be performed to study the effects of the local soil/site conditions (site effects).

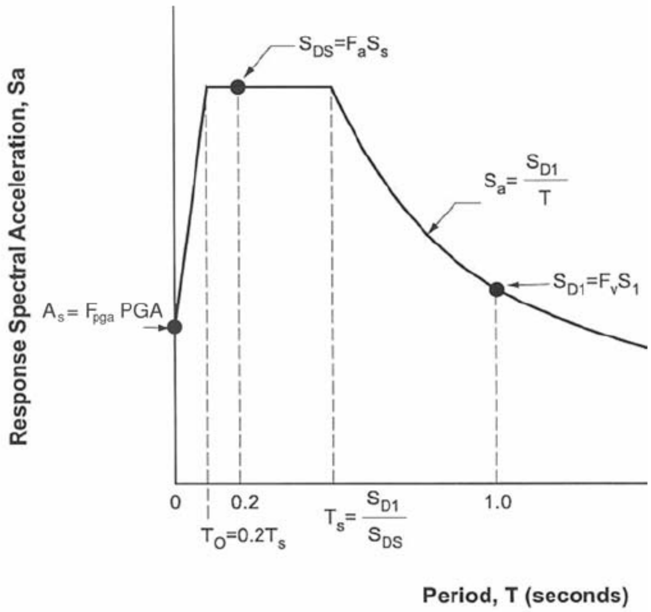


Figure 13-7 Design Response Spectra Constructed Using the NEHRP Procedure

It should be noted that while the design response spectra are commonly used for the seismic design and analysis of above-ground structures such as bridges and buildings, they are not as useful in the seismic evaluation for underground structure. This is because response spectra are more relevant for evaluating the inertial response effect of above-ground structures while for underground structures, ground strains or ground displacements are the governing factor. Nevertheless, design response spectra effectively establish the ground motion shaking intensity level and can be used for deriving other ground motion parameters that are useful and relevant for underground structures. For example, using the design spectral acceleration at 1.0 sec (S_{D1}), PGV can be estimated using the empirical correlation discussed above (Equation 13-1). In addition, design response spectra can also be used as the target spectra for generating the design ground motion time histories which in turn can be used in seismic analysis for underground structures if more refined numerical analysis is required.

Ground Motion Time histories and Spatially Varying Ground Motion Effects: The developed time histories should match the target design response spectra and have characteristics that are representative of the seismic environment of the site and the local site conditions. Characteristics of the seismic environment of the site to be considered in selecting time-histories include: tectonic environment (e.g., subduction zone; shallow crustal faults in WUS or similar crustal environment; CEUS or similar crustal environment); earthquake magnitude; type of faulting (e.g., strike-slip; reverse; normal); seismic-source-to-site distance; local site conditions; and design or expected ground-motion characteristics (e.g., design response spectrum; duration of strong shaking; and special ground-motion characteristics such as near-fault characteristics).

It is desirable to select time-histories that have been recorded under conditions similar to the seismic conditions (as described above) at the site, but compromises are usually required because of the multiple attributes of the seismic environment and the limited data bank of recorded time-histories. Selection of time-histories having similar earthquake magnitudes and distances, within reasonable ranges, are especially important parameters because they have a strong influence on response spectral content, response spectral shape, duration of strong shaking, and near-source ground-motion characteristics.

For long structures such as tunnels, different ground motions may be encountered by different parts of the structure. Thus, it is sometime necessary for the tunnel to be evaluated for the spatially varying ground motions effects, particularly when the longitudinal response of the tunnel is of concern (refer to discussions in Section 13.5.2). In this case the differential displacements and force buildup along the length of the tunnel could be induced due to the spatially varying ground motion effects. In deriving the spatially varying ground motion time histories, as a minimum the following factors should be taken into considerations:

- Local soil site effect
- Wave traveling/passage effect
- Extended source effect
- Near-field effect.

Ground Motion Parameters Attenuation with Depth: The ground motions parameters discussed above are typically established at ground surface. Tunnels, however, are generally constructed at some depth below the ground surface. For seismic evaluation of the tunnel structure, the ground motion parameters should be derived at the elevation of the tunnel. Because ground motions generally decrease with depth below the ground surface, these parameters generally have lower values than estimated for ground surface motions (e.g., Chang et al., 1986). The ratios of ground motion values at tunnel depths to those at the ground surface may be taken as the ratios summarized in Table 13-1 unless lower values are justified based on site-specific assessments.

For more accurate assessment of the ground motion parameters at depth, site-specific dynamic site response analysis should be performed to account for detailed subsurface conditions and site geometry. Results from the dynamic site response analysis would provide various aspects of ground motion parameters as a function of depth (in a one-dimensional site response analysis) or as a function of spatial coordinates (in a two- or three-dimensional site response analysis).

Table 13-1 Ground Motion Attenuation with Depth

Tunnel Depth (m)	Ratio Of Ground Motion At Tunnel Depth To Motion At Ground Surface
≤ 6	1.0
6 -15	0.9
15 -30	0.8
≥ 30	0.7

13.3 FACTORS THAT INFLUENCE TUNNEL SEISMIC PERFORMANCE

The main factors influencing tunnel seismic performance generally can be summarized as (1) seismic hazard, (2) geologic conditions, and (3) tunnel design, construction, and condition. Each of these factors is briefly described in the following sections.

13.3.1 Seismic Hazard

In a broad sense, earthquake effects on underground tunnel structures can be grouped into two categories: (1) ground shaking, and (2) ground failure. Based on tunnel performance records during past earthquakes, the damaging effects of ground failure on tunnels are significantly greater than the ground shaking effects.

Ground Shaking: Ground shaking refers to the vibration of the ground produced by seismic waves propagating through the earth's crust. The area experiencing this shaking may cover hundreds of square miles in the vicinity of the fault rupture. The intensity of the shaking attenuates with distance from the fault rupture. Ground shaking motions are composed of two different types of seismic waves, each with two sub-types, described as follows:

- Body waves traveling within the earth's material. They may be either longitudinal P waves or transverse shear S waves and they can travel in any direction in the ground.
- Surface waves traveling along the earth's surface. They may be either Rayleigh waves or Love waves.

As the ground is deformed by the traveling waves, any tunnel structure in the ground will also be deformed, since tunnel structures are constrained by the surrounding medium (soil or rock). As long as the ground (i.e., the surrounding medium) is stable, the structures cannot move independently of the ground. Therefore, the design and analysis of underground structures is based on ground deformations/strains rather than ground acceleration values. If the magnitude of ground deformation during earthquakes is small, the seismic effect on tunnels is negligible. For example, there is generally little concern for tunnel sections constructed in reasonably competent rock because the seismically induced deformations/strains in rock are generally very small, except when shear/fault zones are encountered or when there are large loosened rock pieces behind the lining. In loose or soft soil deposits, on the other hand, the soil deformation developed during the design earthquake(s) should be estimated and used for the structure's design and analysis. In general the potential effects of ground shaking range from minor cracking of a concrete liner to collapse of the liner and major caving of geologic materials into the tunnel.

Ground Failure: Ground failure broadly includes various types of ground instability such as fault rupture, tectonic uplift and subsidence, landsliding, and soil liquefaction. Each of these hazards may be potentially catastrophic to tunnel structures, although the damages are usually localized. Design of a tunnel structure against ground instability problems is often possible, although the cost may be high.

If an active fault crosses the tunnel alignment, there is a hazard of direct shearing displacement through the tunnel in the event of a moderate to large magnitude earthquake. Such displacements may range from a few inches to greater than ten feet and, in many cases, may be concentrated in a narrow zone along the fault. Fault rupture can and has had very damaging effects on tunnels. Tectonic uplift and subsidence can have similar damaging effects to fault rupture, if the uplift/subsidence movements cause sufficient differential deformation of the tunnel.

Landsliding through a tunnel, whether statically or seismically induced, can result in large, concentrated shearing displacements and either full or partial collapse of tunnel cross sections. Landslide potential is greatest when a preexisting landslide mass intersects the tunnel. A statically stable landslide mass may be activated by earthquake shaking. The hazard of landsliding is usually greatest in shallower parts of a tunnel alignment and at tunnel portals.

For tunnels located in soils below the groundwater table, there could be a potential for liquefaction if loose to medium-dense cohesionless soils (sands, silts, gravels) are adjacent to the tunnel. Potential effects of liquefaction of soils adjacent to a tunnel include: (a) increased lateral pressures on the lining or walls of the tunnel, which could lead to failure of the lining or walls depending on their design; (b) flotation or sinking of a tunnel embedded in liquefied soil, depending on the relative weight of the tunnel and the soils replaced by the tunnel; and (c) lateral displacements of a tunnel if there is a free face toward which liquefied soil can move and/or if the tunnel is constructed below sloping ground.

13.3.2 Geologic Conditions

Other unfavorable geologic conditions could lead to unsatisfactory seismic tunnel performance unless recognized and adequately accounted for in the tunnel design and construction. Unfavorable geologic conditions include: soft soils; rocks with weak planes intersecting a tunnel, such as shear zones or well developed weak bedding planes and well developed joint sets that are open or filled with weathered and decomposed rock; failures encountered during tunnel construction that may have further weakened the geologic formations adjacent to a tunnel (e.g., cave-ins or running ground leaving incompletely filled voids or loosened rock behind a lining; squeezing ground with relatively low static factor of safety against lining collapse); and adjacent geologic units having major contrasts in stiffness that can lead to stress concentrations or differential displacement.

13.3.3 Tunnel Design, Construction, and Condition

Elements of tunnel design, construction, and condition that may influence tunnel seismic behavior include:

1. Whether seismic loadings and behavior were explicitly considered in tunnel design
2. The nature of the tunnel lining and support system (e.g., type of lining, degree of contact between lining/support systems and geologic material, use of rock bolts and dowels)
3. Junctions of tunnels with other structures
4. History of static tunnel performance in terms of failures and cracking or distortion of lining/support system
5. Current condition of lining/support system, such as degree of cracking of concrete and deterioration of concrete or steel materials over time.

In evaluating an existing tunnel in the screening stage or in a more detailed evaluation, or in designing retrofit measures, it is important to obtain as complete information as possible on the tunnel design, construction, and condition and the geologic conditions along the tunnel alignment. To obtain this information, the design and evaluation team should review the design drawings and design studies, as-built drawings, construction records as contained in the construction engineer daily reports and any special reports, maintenance and inspection records, and geologic and geotechnical reports and maps. Special inspections and investigations may be needed to adequately depict the existing conditions and determine reasons for any distress to the tunnel.

13.4 SEISMIC PERFORMANCE AND SCREENING GUIDELINES OF TUNNELS

13.4.1 Screening Guidelines Applicable to All Types of Tunnels

There are certain conditions that would clearly indicate a potentially significant seismic risk to a bored tunnel, cut-and-cover tunnel, or submerged tube and thus require more detailed evaluations. These conditions include:

- An active fault intersecting the tunnel;
- A landslide intersecting the tunnel, whether or not the landslide is active;
- Liquefiable soils adjacent to the tunnel, and
- History of static distress to the tunnel (e.g., local collapses, large deformations, cracking or spalling of the liner due to earth movements), unless retrofit measures were taken to stabilize the tunnel.

In addition to the above, detailed seismic evaluations should also be conducted for tunnels that are considered lifeline structures (important and critical structures) that must be usable or remain open to traffic immediately after the earthquake. Transit tunnels in metropolitan areas are often considered as critical/lifeline structures and, therefore, warrant detailed seismic evaluations.

13.4.2 Additional Screening Guidelines for Bored Tunnels

If the above conditions do not exist, then the risk to a bored tunnel is a function of the tunnel design and construction, the characteristics of the geologic media, and the level of ground shaking. In this section, additional screening guidelines are presented considering these factors and empirical observations of tunnel performance during earthquakes.

It should be noted that although not as damaging as ground failure effects, ground shaking effect alone (i.e., in the absence of ground failure) has resulted in moderate to major damage to many tunnels in earthquakes. Figure 13-8 shows a highway tunnel experiencing lining falling off from tunnel crown under the ground shaking effect during the 2004 Niigata Earthquake in Japan. In another incident, the 1999 Koceali Earthquake in Turkey caused the collapse of two tunnels (the Bolu Tunnels) constructed using NATM method (15 m arch high and 16 m wide). At the time of the earthquake, the collapsed section of the tunnel had been stabilized with steel rib, shotcrete, and anchors.

Figure 13-8 Highway Tunnel Lining Falling from Tunnel Crown – 2004 Niigata Earthquake, Japan

Figure 13-9 presents a summary of empirical observations of the effects of seismic ground shaking on the performance of bored/mined tunnels. The figure is from the study by Power et al. (1998), which updates earlier presentations of tunnel performance data by Dowding and Rozen (1978), Owen and Scholl (1981), and Sharma and Judd (1991). The data are for damage due only to shaking; damage that was definitely or probably attributed to fault rupture, landsliding, and liquefaction is not included. The data are for bored/mined tunnels only; data for cut-and-cover tunnels and submerged tubes are not included in Figure 13-9.

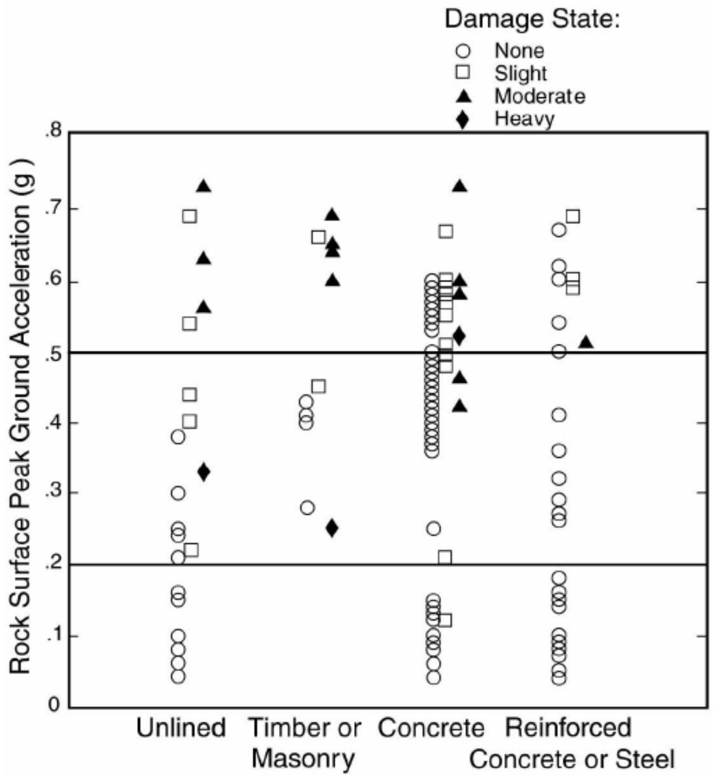


Figure 13-9 Summary of Observed Bored/Mined Tunnel Damage under Ground Shaking Effects (Power et al., 1998)

Figure 13-9 incorporates observations for 192 tunnels from ten moderate to large magnitude earthquakes (moment magnitude MW 6.6 to 8.4) in California, Japan, and Alaska. Ninety-four of the observations are from the moment magnitude M_W 6.9 1995 Kobe, Japan, earthquake. This earthquake produced by far the most observations for moderate to high levels of shaking (estimated peak ground accelerations, PGA, at ground surface above the tunnels in the range of about 0.4 g to 0.6 g for the Kobe data). Peak ground accelerations in Figure 13-9 are estimated for actual or hypothetical outcropping rock conditions at ground surface above the tunnel. Other observations are from moderate to large (M_W 6.7 to 8.4) earthquakes in California and Japan. Figure 13-9 shows the level of damage induced in tunnels with different types of linings subjected to the indicated levels of ground shaking. Damage was categorized into four states: none for no observable damage; slight for minor cracking and spalling; moderate for major cracking and spalling, falling of pieces of lining and rocks; and heavy for major cave-ins, blockage, and collapse. The figure indicates the following trends:

- For PGA equal to or less than 0.2 g, ground shaking caused essentially no damage in tunnels.
- For PGA in the range of 0.2 g to 0.5 g, there are some instances of damage ranging from slight to heavy. Note that the three instances of heavy damage are all from the 1923 Kanto, Japan, earthquake. For the 1923 Kanto earthquake observation with PGA equal to 0.25 g shown on Figure 13-9, the investigations for this tunnel indicated the damage may have been due to landsliding. For the other two Kanto earthquake observations, collapses occurred in the shallow portions of the tunnels.

- For PGA exceeding about 0.5 g, there are a number of instances of slight to moderate damage (and one instance of heavy damage noted above for the Kanto earthquake).
- Tunnels with stronger linings appear to have performed better, especially those tunnels with reinforced concrete and/or steel linings.

The trends in Figure 13-9 can be used as one guide in assessing the need for further evaluations of the effects of ground shaking on bored/mined tunnels.

13.4.3 Additional Screening Guidelines for Cut-and-Cover Tunnels

Reporting on the seismic performance of shallow cut-and-cover box-like tunnels has been relatively poor in comparison to the performance of bored/mined tunnels. This was especially evident during the 1995 Kobe, Japan, earthquake (O'Rourke and Shiba, 1997; Power et al., 1998). Figure 13-10 and Figure 13-11 show the damage to the center columns of the cut-and-cover tunnels running between Daikai and Nagata Stations during the 1995 Kobe Earthquake.

Figure 13-10 Fracture at Base of Columns of Cut-and-Cover Tunnel between Daikai and Nagata Stations - 1995 Kobe Earthquake, Japan

Figure 13-11 Shear Failure at Top of Columns of Cut-and-Cover Tunnel Between Daikai and Nagata Stations - 1995 Kobe Earthquake, Japan

The 1995 Kobe Earthquake also caused a major collapse of the Daikai subway station which was constructed by cut-and-cover method without specific seismic design provisions. The schematic drawing shown in Figure 13-12 (Iida et al., 1996) shows the collapse experienced by the center columns of the station, which was accompanied by the collapse of the ceiling slab and the settlement of the soil cover by more than 2.5 m.

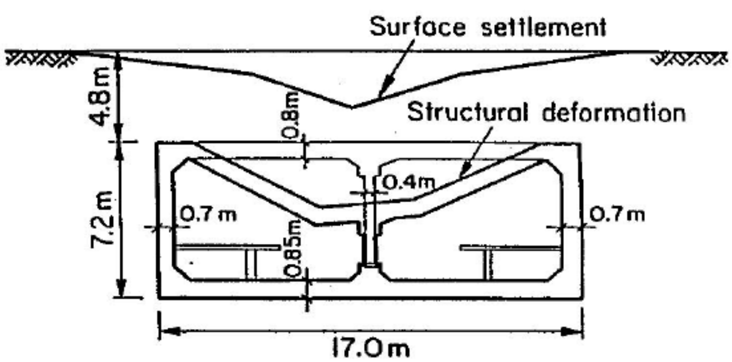


Figure 13-12 Daikai Subway Station Collapse – 1995 Kobe Earthquake, Japan

The relatively poor performance of cut-and-cover tunnels under the ground shaking effect may reflect: (1) relatively softer near-surface geologic materials surrounding these types of structures as compared to the harder materials that often surround bored tunnels at greater depths; (2) higher levels of acceleration at and near the ground surface than at depth (due to tendencies for vibratory ground motions to reduce with depth below the ground surface); and (3) vulnerability of these box-like structures to seismically induced racking deformations of the box cross section (Refer to Figure 13-13 in Section 13.5), unless specifically designed to accommodate these racking deformations. Cut-and-cover tunnels in soil tend to be more vulnerable than those excavated into rock because of the larger soil shear deformations causing the tunnel racking. Tunnels in soft soil may be especially vulnerable. The most important determinant in assessing whether more detailed seismic evaluations of cut-and-cover tunnels are required is whether the original design considered loadings and deformations consistent with the seismic environment and geologic conditions, and especially, whether racking behavior was taken into account in the seismic analysis, design, and detailing of the structure.

13.4.4 Additional Screening Guidelines for Immersed Tubes

Submerged tubes are particularly susceptible to permanent ground movements during seismic shaking. Tubes are typically located at shallow depths and in soft or loose soils. Liquefaction of loose cohesionless soils may cause settlement, uplift (flotation), or lateral spreading. Earthquake shaking may also cause permanent displacement of soft clay soils on sloping ground. Joints connecting tube segments must accommodate the relative displacement of adjacent segments while maintaining a watertight seal. Generally, submerged tubes can be screened out from more detailed evaluations if the original design appropriately considered and analyzed the potential for ground failure modes and if joints have been carefully designed to achieve water tightness.

13.5 SEISMIC EVALUATION PROCEDURES - GROUND SHAKING EFFECTS

Underground tunnel structures undergo three primary modes of deformation during seismic shaking: ovaling/racking, axial and curvature deformations. The ovaling/racking deformation is caused primarily by seismic waves propagating perpendicular to the tunnel longitudinal axis, causing deformations in the plane of the tunnel cross section (Refer to Figure 13-3, Wang, 1993; Owen and Scholl, 1981). Vertically propagating shear waves are generally considered the most critical type of waves for this mode of deformation. The axial and curvature deformations are induced by components of seismic waves that propagate along the longitudinal axis (Refer to Figure 13-14, Wang, 1993; Owen and Scholl, 1981).

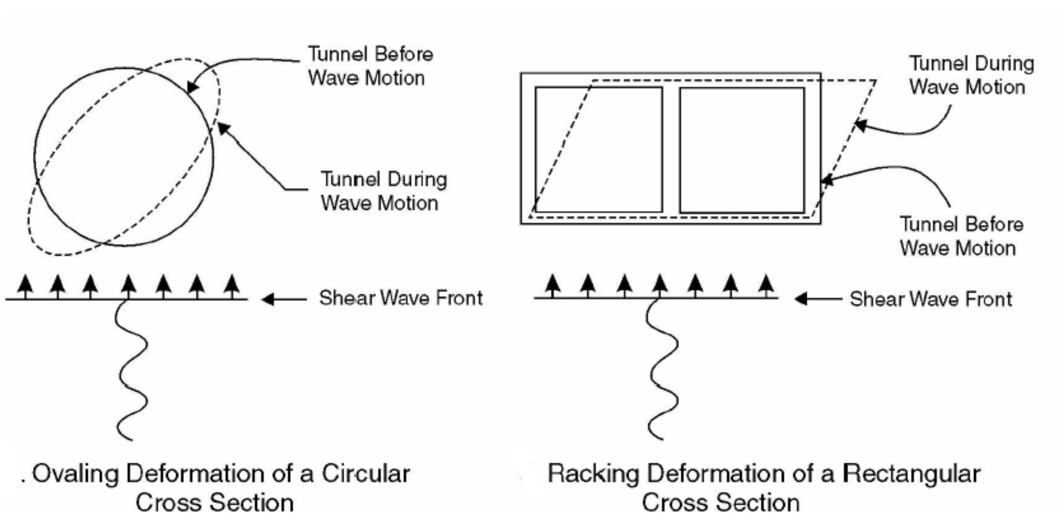


Figure 13-13 Tunnel Transverse Ovaling and Racking Response to Vertically Propagating Shear Waves

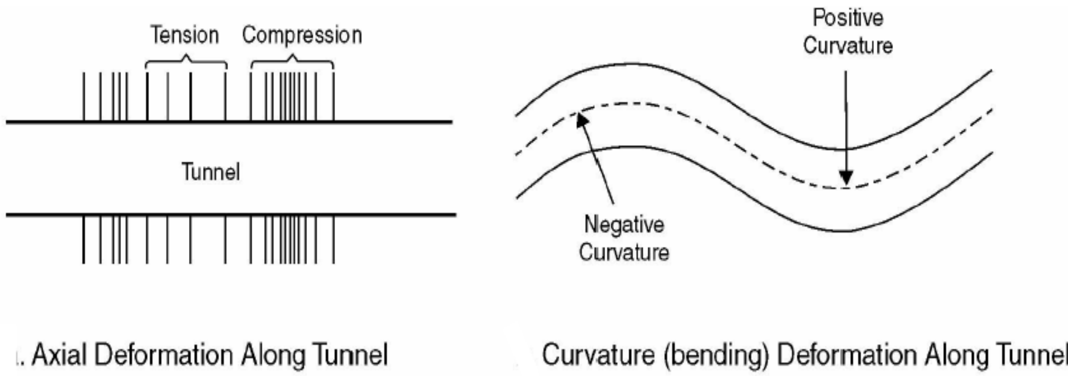


Figure 13-14 Tunnel Longitudinal Axial and Curvature Response to Traveling Waves

13.5.1 Evaluation of Transverse Ovaling/Racking Response of Tunnel Structures

The evaluation procedures for transverse response of tunnel structures can be based on either (1) simplified analytical method, or (2) more complex numerical modeling approach, depending on the degree of complexity of the soil-structure system, subsurface conditions, the seismic hazard level, and the importance of the structures. The numerical modeling approach should be considered in cases where simplified analysis methods are less applicable, more uncertain, or inconclusive, or where a very important structure is located in a severe seismic environment or where case history data indicate relatively higher seismic vulnerability for the type of tunnel, such as rectangular cut-and-cover tunnels in seismically active areas. The numerical modeling approach is further discussed in Section 13.5.1.4.

13.5.1.1 Simplified Procedure for Ovaling Response of Circular Tunnels

This section provides methods for quantifying the seismic ovaling effect on circular tunnel linings. The conventionally used simplified free-field deformation method, discussed first, ignores the soil-structure interaction effects. Therefore its use is limited to conditions where the tunnel structures can be reasonably assumed to deform according to the free-field displacements during earthquakes.

A refined method is then presented in Section 13.5.1.2 that is equally simple but capable of eliminating the drawbacks associated with the free-field deformation method. This refined method - built from a theory that is familiar to most mining/underground engineers - considers the soil-structure interaction effects. Based on this method, a series of design charts are developed to facilitate the design process.

Ovaling Effect: As mentioned earlier, ovaling of a circular tunnel lining is primarily caused by seismic waves propagating in planes perpendicular to the tunnel axis. The results are cycles of additional stress concentrations with alternating compressive and tensile stresses in the tunnel lining. These dynamic stresses are superimposed on the existing static state of stress in the lining. Several critical modes may result (Owen and Scholl, 1981):

- Compressive dynamic stresses added to the compressive static stresses may exceed the compressive capacity of the lining locally.
- Tensile dynamic stresses subtracted from the compressive static stresses reduce the lining's moment capacity, and sometimes the resulting stresses may be tensile.

<u>Free-Field Shear Deformations:</u> As mentioned previously, the shear distortion of ground caused by vertically propagating shear waves is probably the most critical and predominant mode of seismic motions. It causes a circular tunnel to oval and a rectangular underground structure to rack (sideways motion), as shown in Figure 13-13. Analytical procedures by numerical methods are often required to arrive at a reasonable estimate of the free-field shear distortion, particularly for a soil site with variable stratigraphy. Many computer codes with variable degree of sophistication are available (e.g., SHAKE, FLUSH, FLAC, PLAXIS, et al.). The most widely used approach is to simplify the site geology into a horizontally layered system and to derive a solution using one-dimensional wave propagation theory (Schnabel, Lysmer, and Seed, 1972). The resulting free-field shear distortion of the ground from this type of analysis can be expressed as a shear strain distribution or shear deformation profile versus depth.

For a deep tunnel located in relatively homogeneous soil or rock and in the absence of detailed site response analyses, the simplified procedure by Newmark (1968) and Hendron (1985) may provide a reasonable estimate, noting, however, that this method tends to produce more conservative results particularly when the effect of ground motion attenuation with depth (refer to Table 13-1) is ignored. Here, the maximum free-field shear strain, γ_{max}, can be expressed as

$$\gamma_{\max} = \frac{V_S}{C_{se}} \qquad \text{13-3}$$

Where:

V_S = Peak particle velocity
C_{se} = Effective shear wave propagation velocity

The effective shear wave velocity of the vertically propagating shear wave, C_{se}, should be compatible with the level of the shear strain that may develop in the ground at the elevation of the tunnel under the design earthquake shaking. The values of C_{se} can be estimated by making proper reduction (to account for the strain-level dependent effect) from the small-strain shear wave velocity, C_s, obtained from in-situ testing (such as using the cross-hole, down-hole, and P-S logging techniques). For rock, the ratio of C_{se}/C_s can be assumed equal to 1.0. For stiff to very stiff soil, C_{se}/C_s may range from 0.6 to 0.9. Alternatively, site specific response analyses can be performed for estimating C_{se}. Site specific response analyses should be performed for estimating C_{se} for tunnels embedded in soft soils

An equation relating the effective propagation velocity of shear waves to effective shear modulus, G_m, is expressed as:

$$C_{se} = \sqrt{\frac{G_m}{\rho}} \qquad \text{13-4}$$

Where:

ρ = Mass density of the ground

An alternative simplified method for calculating the free-field ground shear strain, γ_{max}, is by dividing the earthquake-induced shear stresses (τ_{max}) by the shear stiffness (i.e., the strain-compatible effective shear modulus, G_m). This method is especially suitable for tunnels with shallow burial depths.

In this simplified method the maximum free-field ground shear strain is calculated using the following equation:

$$\gamma_{max} = \frac{\tau_{max}}{G_m} \qquad 13\text{-}5$$

$$\tau_{max} = (PGA/g)\ \sigma_v\ R_d \qquad 13\text{-}6$$

$$\sigma_v = \gamma_t\ (H+D) \qquad 13\text{-}7$$

Where:

G_m	= Effective strain-compatible shear modulus of ground surrounding tunnel (ksf)
τ_{max}	= Maximum earthquake-induced shear stress (ksf)
σ_v	= Total vertical soil overburden pressure at invert elevation of tunnel (ksf)
γ_t	= Total soil unit weight (kcf)
H	= Soil cover thickness measured from ground surface to tunnel crown (ft)
D	= Height of tunnel (or diameter of circular tunnel) (ft)
R_d	= Depth dependent stress reduction factor; can be estimated using the following relationships:

$R_d = 1.0 - 0.00233z$ for $z < 30$ ft
$R_d = 1.174 - 0.00814z$ for 30 ft $< z <$ 75 ft
$R_d = 0.744 - 0.00244z$ for 75 ft $< z <$ 100 ft
$R_d = 0.5$ for $z > 100$ ft

Where:

z = the depth (ft) from ground surface to the invert elevation of the tunnel and is represented by z = (H+D).

<u>Lining Conforming to Free-Field Shear Deformations:</u> When a circular lining is assumed to oval in accordance with the deformations imposed by the surrounding ground (e.g., shear), the lining's transverse sectional stiffness is completely ignored. This assumption is probably reasonable for most circular tunnels in rock and in stiff soils, because the lining stiffness against distortion is low compared with that of the surrounding medium. Depending on the definition of "ground deformation of surrounding medium," however, a design based on this assumption may be overly conservative in some cases and non-conservative in others. This will be discussed further below.

Shear distortion of the surrounding ground, for this discussion, can be defined in two ways. If the non-perforated ground in the free-field is used to derive the shear distortion surrounding the tunnel lining, the lining is to be designed to conform to the maximum diameter change, $\Delta D_{free\text{-}field}$, shown in the top of Figure 13-15.

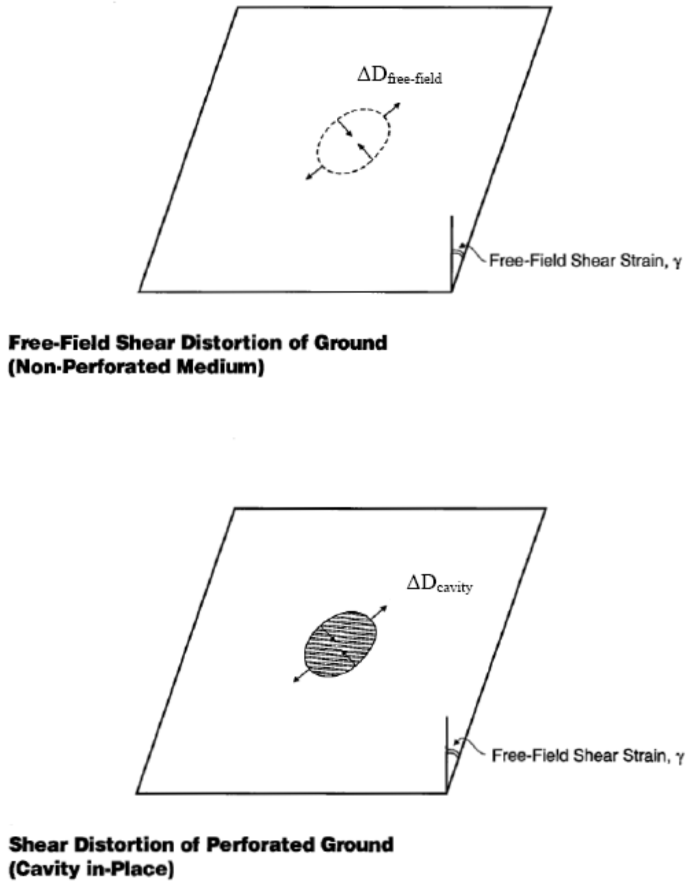


Figure 13-15 Shear Distortion of Ground – Free-Field Condition vs Cavity In-Place Condition

The maximum diametric change of the lining for this case can be derived as:

$$\Delta D_{free-field} = \pm(\gamma_{\max}/2)D \qquad 13\text{-}8$$

Where:

D = the diameter of the tunnel
γ_{max} = the maximum free-field shear strain

On the other hand, if the ground deformation is derived by assuming the presence of a cavity due to tunnel excavation (bottom of Figure 13-15, for perforated ground), then the lining is to be designed according to the diametric strain expressed as:

$$\Delta D_{cavity} = \pm 2\gamma_{\max}(1-\nu_m)D \qquad 13\text{-}9$$

Where:

ν_m = the Poisson's Ratio of the medium

Equations 13-8 and 13-9 both assume the absence of the lining. In other words, tunnel-ground interaction is ignored.

Comparison between Equations 13-8 and 13-9 shows that the perforated ground deformation would yield a much greater distortion than the free-field case (non-perforated ground). For a typical ground medium, the difference could be as much as three times. Based on the assumptions made, some preliminary conclusions can be drawn as follows:

- Equation 13-9, for the perforated ground deformation, should provide a reasonable estimate for the deformation of a lining that has little stiffness (against distortion) in comparison to that of the medium.
- Equation 13-8, for the free-field ground deformation, on the other hand, should provide a reasonable result for a lining with a distortion stiffness close or equal to the surrounding medium.

Based on the discussions above, it can be further suggested that a lining with a greater distortion stiffness than the surrounding medium should experience a lining distortion even less than the free-field deformation. This latest case may occur when a tunnel is built in soft to very soft soils. It is therefore clear that the relative stiffness between the tunnel and the surrounding ground (i.e., soil-structure interaction effect) plays an important role in quantifying tunnel response during the seismic loading condition. This effect will be discussed next.

<u>Importance of Lining Stiffness- Compressibility and Flexibility Ratios:</u> To quantify the relative stiffness between a circular lining and the medium, two ratios designated as the compressibility ratio, C, and the flexibility ratio, F (Hoeg, 1968, and Peck et al., 1972) are defined by the following equations:

Compressibility Ratio:

$$C = \frac{E_m(1-\nu_l^2)R_l}{E_l t(1+\nu_m)(1-2\nu_m)} \qquad 13\text{-}10$$

Flexibility Ratio:

$$F = \frac{E_m(1-\nu_l^2)R_l^3}{6E_l I_{l,1}(1+\nu_m)} \qquad 13\text{-}11$$

Where:

E_m = Strain-compatible elastic modulus of the surrounding ground

ν_m = Poisson's ratio of the surrounding ground

R_l = Nominal radius of the tunnel lining

ν_l = Poisson's ratio of the tunnel Lining

$I_{l,1}$ = Moment of inertia of lining per unit width of tunnel along the tunnel axis.

t_l = The thickness of the lining

Of these two ratios, it often has been suggested that the flexibility ratio is the more important because it is related to the ability of the lining to resist distortion imposed by the ground. As will be discussed later, the compressibility ratio also has a significant effect on the lining thrust response.

For most circular tunnels encountered in practice, the flexibility ratio, F, is likely to be large enough (say, F>20) so that the tunnel-ground interaction effect can be ignored (Peck, 1972). It is to be noted that F > 20 suggests that the ground is about 20 times stiffer than the lining. In these cases, the distortions to be experienced by the lining can be reasonably assumed to be equal to those of the perforated ground (i.e., ΔD_{cavity}).

This rule of thumb procedure may present some design problems when a very stiff structure is surrounded by a very soft soil. A typical example would be to construct a very stiff immersed tube in a soft lake or river bed deposit. In this case the flexibility ratio is very low, and the stiff tunnel lining could not be realistically designed to conform to the deformations imposed by the soft ground. The tunnel-ground interaction effect must be considered in this case to achieve a more efficient design.

In the following section a refined procedure taking into account the tunnel-ground interaction effect is presented to provide a more accurate assessment of the seismic ovaling effect on a circular lining.

13.5.1.2 Analytical Lining-Ground Interaction Solutions for Ovaling Response of Circular Tunnels

Closed form analytical solutions have been proposed (Wang, 1993) for estimating ground-structure interaction for circular tunnels under the seismic loading conditions. These solutions are generally based on the assumptions that:

- The ground is an infinite, elastic, homogeneous, isotropic medium.
- The circular lining is generally an elastic, thin walled tube under plane strain conditions.
- Full-slip or no-slip conditions exist along the interface between the ground and the lining.

The expressions of these lining responses are functions of flexibility ratio and compressibility ratio as presented previously in Equations 13-10 and 13-11. The expressions for maximum thrust, T_{max}, bending moment, M_{max}, and diametric strain, $\Delta D/D$, can be presented in the following forms:

$$M_{\max} = \pm \frac{1}{6} K_1 \frac{E_m}{(1+\nu_m)} R_l^2 \gamma_{\max} \qquad 13\text{-}12$$

$$T_{\max} = \pm K_2 \frac{E_m}{2(1+\nu_m)} R_l \gamma_{\max} \qquad 13\text{-}13$$

$$\Delta D_{\max} / D = \pm \tfrac{1}{3} K_1 F \gamma_{\max} \qquad 13\text{-}14$$

$$K_1 = \frac{12(1-\nu_m)}{2F + 5 - 6\nu_m} \qquad 13\text{-}15$$

$$K_2 = 1 + \frac{F[(1-2\nu_m)-(1-2\nu_m)C] - \frac{1}{2}(1-2\nu_m)^2 C + 2}{F[(3-2\nu_m)+(1-2\nu_m)C] + C[\frac{5}{2} - 8\nu_m + 6\nu_m^2] + 6 - 8\nu_m} \qquad \text{13-16}$$

K_1 and K_2 are defined herein as lining response coefficients. The earthquake loading parameter is represented by the maximum shear strain induced in the ground (free-field), γ_{max}, which may be obtained through a simplified approach (such as Equation 13-15 or 13-16), or by performing a site-response analysis.

The resulting bending moment induced maximum fiber strain, ε_m, and the axial force (i.e., thrust) induced strain, ε_T, can be derived as follows:

$$\varepsilon_m = \pm \frac{1}{6} K_1 \frac{E_m}{(1+\nu_m)} R_l^2 \frac{\gamma_{\max} t_l}{2 E_l I_l} \qquad \text{13-17}$$

$$\varepsilon_T = \pm K_2 \frac{E_m}{2(1+\nu_m)} R_l \frac{\gamma_{\max}}{E_l t_l} \qquad \text{13-18}$$

To ease the design process, Figure 13-16 shows the lining response coefficient, K_1, as a function of flexibility ratio and Poisson's Ratio of the ground. The design charts showing the lining coefficient K_2, primarily used for the thrust response evaluation, are presented in Figure 13-17, Figure 13-18, and Figure Figure 13-19 for Poisson's Ratio values of 0.2, 0.35 and 0.5, respectively.

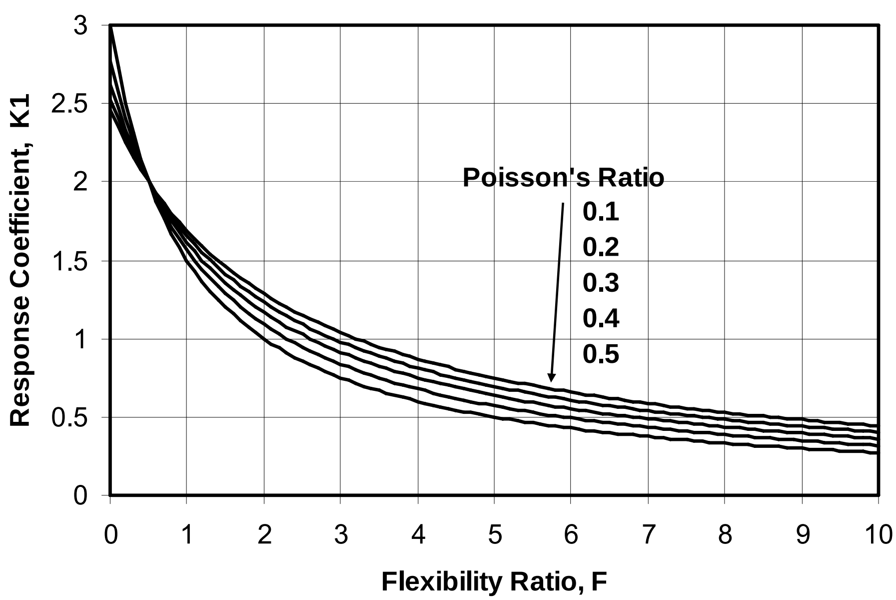


Figure 13-16 Lining Response Coefficient, K_1 (Full-Slip Interface Condition)

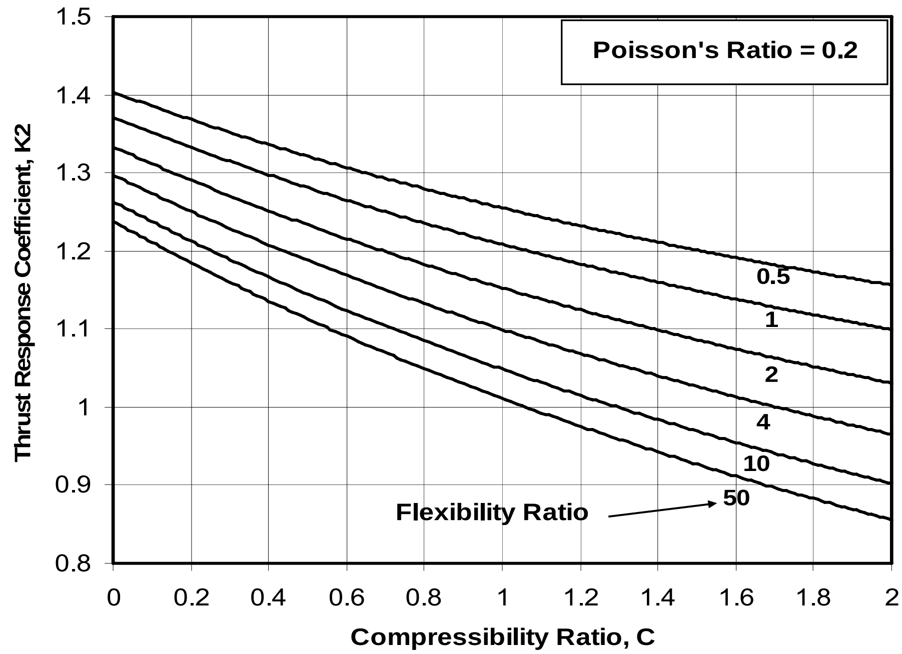


Figure 13-17 Lining Response Coefficient, K_2, for Poisson's Ratio = 0.2 (No-Slip Interface Condition)

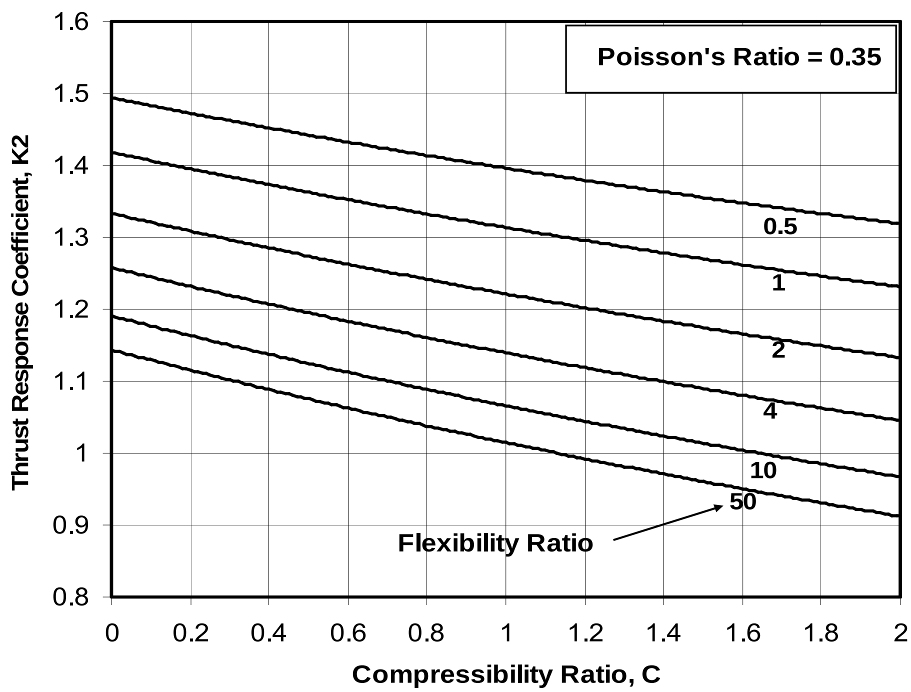


Figure 13-18 Lining Response Coefficient, K_2, for Poisson's Ratio = 0.35 (No-Slip Interface Condition)

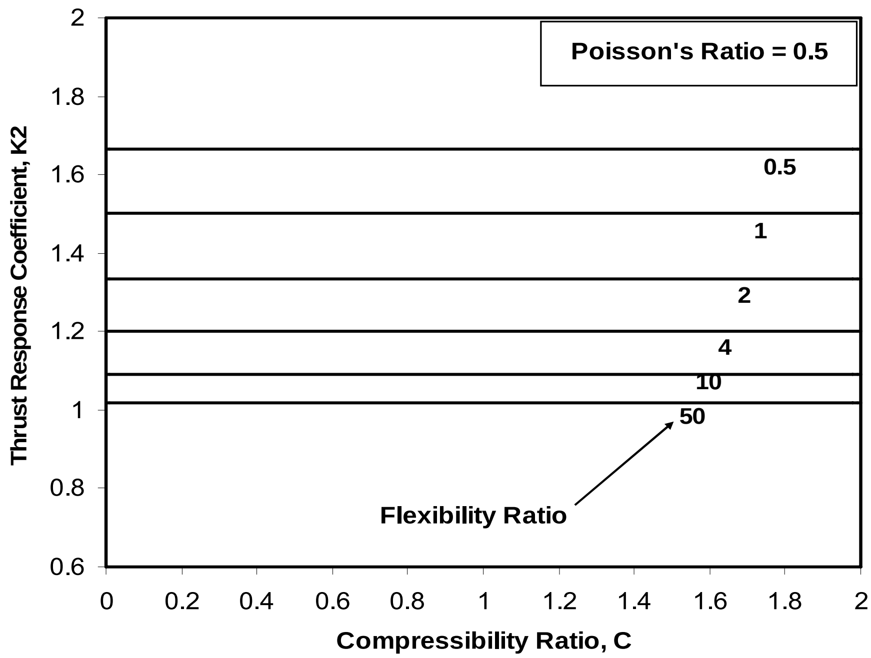


Figure 13-19 Lining Response Coefficient, K_2, for Poisson's Ratio = 0.5 (No-Slip Interface Condition)

It should be noted that the solutions in terms of M_{max}, ΔD_{max}, and ε_m provided herein are based on the full-slip interface assumption. For the maximum thrust response T_{max} the interface conditions is assumed to be no-slip. These assumptions were adopted because full-slip condition produces more conservative results for M_{max} and ΔD_{max}, while no-slip condition is more conservative for T_{max}. During an earthquake, in general, slip at interface is a possibility only for tunnels in soft soils, or when seismic loading intensity is severe. For most tunnels, the condition at the interface is between full-slip and no-slip. In computing the forces and deformations in the lining, it is prudent to investigate both cases and the more critical one should be used in design.

The conservatism described above is desirable to offset the potential underestimation of lining forces resulting from the use of equivalent static model in lieu of the dynamic loading condition. Previous studies suggest that a true dynamic solution would yield results that are 10 to 15 percent greater than an equivalent static solution, provided that the seismic wavelength is at least about 8 times greater than the width of the excavation (cavity). Therefore, the full-slip model is recommended in evaluating the moment and deflection response (i.e., Figure 13-16 and Equation 13-15) of a circular tunnel lining.

Using the full-slip condition, however, would significantly underestimate the maximum thrust, T_{max}, under the seismic simple shear condition. Therefore, it is recommended that the no-slip interface assumption be used in assessing the lining thrust response (Equation13-16).

Effective Lining Stiffness: The results presented above are based on the assumption that the lining is a monolithic and continuous circular ring with intact, elastic properties. Many circular tunnels are constructed with bolted or unbolted segmental lining. Besides, a concrete lining subjected to bending and thrust often cracks and behaves in a nonlinear fashion. Therefore, in applying the results presented herewith, the effective (or, equivalent) stiffness of the lining should be used. Some simple and approximate methods accounting for the effect of joints on lining stiffness can be found in the literature

- Monsees and Hansmire (1992) suggested the use of an effective lining stiffness that is one-half of the stiffness for the full lining section.
- Analytical studies by Paul, et al., (1983) suggested that the effective stiffness be from 30 to 95 percent of the intact, full-section lining.
- Muir Wood (1975) and Lyons (1978) examined the effects of joints in precast concrete segmental linings and showed that for a lining with "n" segments, the effective stiffness of the ring was:

$$I_e = I_j + \left(\frac{4}{n}\right)^2 I \qquad \text{13-19}$$

Where:

$I_e < I$ and $n > 4$
I = Lining stiffness of the intact, full-section
I_j = Effective stiffness of lining at joint
I_e = Effective stiffness of lining

13.5.1.3 Analytical Lining-Ground Interaction Solutions for Racking Response of Rectangular Tunnels

General: Shallow depth transportation tunnels are often of rectangular shape and are often built using the cut-and-cover method. Usually the tunnel is designed as a rigid frame box structure. From the seismic design standpoint, these box structures have some characteristics that are different from those of the bored circular tunnels, besides the geometrical aspects. The implications of three of these characteristics for seismic design are discussed below.

First, cut-and-cover tunnels are generally built at shallow depths in soils where seismic ground deformations and the shaking intensity tend to be greater than at deeper locations, due to the lower stiffness of the soils and the site amplification effect. As discussed earlier, past tunnel performance data suggest that tunnels built with shallow soil overburden cover tend to be more vulnerable to earthquakes than deep ones.

Second, a box frame usually does not transmit the static loads as efficiently as a circular lining, resulting in much thicker walls and slabs for the box frame. As a result, a rectangular tunnel structure is usually stiffer than a circular tunnel lining in the transverse direction and less tolerant to distortion. This characteristic, along with the potential large seismic ground deformations that are typical for shallow soil deposits, makes the soil-structure interaction effect particularly important for the seismic design of cut-and-cover rectangular tunnels, including those built with the sunken/immersed tube method.

Third, typically soil is backfilled above the structure and possibly between the in-situ medium and the structure. Often, the backfill soil may consist of compacted material having different properties than the in-situ soil. The properties of the backfill soil as well as the in-situ medium should be properly accounted for in the design and analysis. The effect of backfill, however, cannot be accounted for using analytical closed-form solutions. Instead, more complex numerical analysis is required for solving this problem if the effect of backfill is considered significant in evaluating seismic response of a cut-and-cover tunnel.

The evaluation procedures presented in this section are based on simplified analytical method. The more refined numerical modeling approach is discussed in Section 13.5.1.4.

Racking Effect: During earthquakes a rectangular box structure in soil or in rock will experience transverse racking deformations (sideways motion) due to the shear distortions of the ground, in a manner similar to the ovaling of a circular tunnel discussed in Section 13.5.1.1. The racking effect on the structure is similar to that of an unbalanced loading condition.

The external forces the structure is subjected to are in the form of shear stresses and normal pressures all around the exterior surfaces of the box. The magnitude and distribution of these external earth forces are complex and difficult to assess. The end results, however, are cycles of additional internal forces and stresses with alternating direction in the structure members. These dynamic forces and stresses are superimposed on the existing static state of stress in the structure members. For rigid frame box structures, the most critical mode of potential damage due to the racking effect is the distress at the top and bottom joints (refer to Figure 13-1, Figure 13-11, Figure 13-12 and Figure 13-13).

Realizing that the overall effect of the seismically induced external earth loading is to cause the structure to rack, it is more reasonable to approach the problem by specifying the loading in terms of deformations. The structure design goal, therefore, is to ensure that the structure can adequately absorb the imposed racking deformation (i.e., the deformation method), rather than using a criterion of resisting a specified dynamic earth pressure (i.e., the force method). The focus of the remaining sections of this chapter, therefore, is on the method based on seismic racking deformations.

Free-Field Racking Deformation Method It has been proposed in the past that a rectangular tunnel structure be designed by assuming that the amount of racking imposed on the structure is equal to the "free-field" shear distortions of the surrounding medium, as illustrated in Figure 13-20 (i.e., $\Delta_{free\text{-}field} = \Delta_s$). The racking stiffness of the structure is ignored with this assumption.

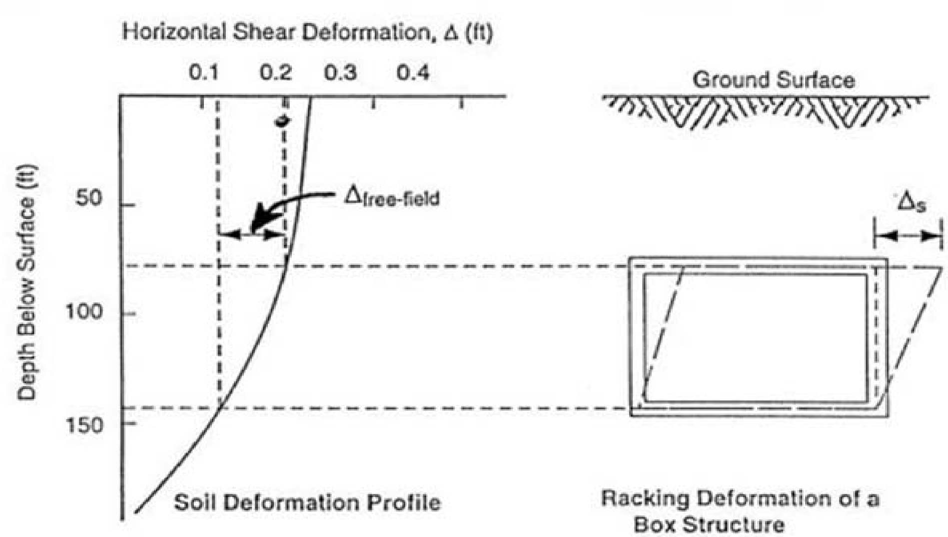


Figure 13-20 Soil Deformation Profile and Racking Deformation of a Box Structure

The free-field deformation method serves as a simple and effective design tool when the seismically induced ground distortion is small, for example when the shaking intensity is low or the ground is very stiff. Given these conditions, most practical structural configurations can easily absorb the ground distortion without being distressed. The method is also a realistic one when the racking stiffness of the structure is comparable to that of its surrounding medium.

It has been reported (Wang, 1993), however, that this simple procedure could lead to overly conservative design (i.e., when $\Delta_{free\text{-}field} > \Delta_s$) or un-conservative design (i.e., when $\Delta_{free\text{-}field} < \Delta_s$), depending on the relative stiffness between the ground and the structure. The overly conservative cases generally occur in soft soils. Seismically induced free-field ground distortions are generally large in soft soils, particularly when they are subjected to amplification effects. Ironically, rectangular box structures in soft soils are generally designed with stiff configurations to resist the static loads, making them less tolerant to racking distortions. Imposing free-field deformations on a structure in this situation is likely to result in unnecessary conservatism, as the stiff structure may deform less than the soft ground.

On the other hand, the un-conservative cases arise when the shear stiffness of the ground is greater than the racking stiffness of the structures – a behavior similar to that described for the ovaling of circular tunnel (Section 13.5.1.1). To more accurately quantify the racking response of rectangular tunnel structures a rational procedure accounting for the tunnel-ground interaction effect is presented in the following section.

<u>Tunnel-Ground Interaction Analysis</u>: Although closed-form solutions accounting for soil-structure interaction, such as those presented in Section 13.5.1.1, are available for deep circular lined tunnels, they are not readily available for rectangular tunnels due primarily to the highly variable geometrical characteristics typically associated with rectangular tunnels. Complex earthquake induced stress-strain conditions is another reason as most of the rectangular tunnels are built using the cut-and-cover method at shallow depths, where seismically induced ground distortions and stresses change significantly with depth.

To develop a simple and practical design procedure, Wang (1993) performed a series of dynamic soil-structure interaction finite element analyses. In this study, the main factors that may potentially affect the dynamic racking response of rectangular tunnel structures were investigated. These factors include:

- Relative Stiffness between Soil and Structure. Based on results derived for circular tunnels (see 13.5.1.1), it was anticipated that the relative stiffness between soil and structure is the dominating factor governing the soil/structure interaction. A series of analyses using ground profiles with varying properties and structures with varying racking stiffness was conducted for parametric study purpose. A special case where a tunnel structure is resting directly on stiff foundation materials (e.g., rock) was also investigated.
- Structure Geometry. Five different types of rectangular structure geometry were studied, including one-barrel, one-over-one two-barrel, and one-by-one twin-barrel tunnel structures.
- Input Earthquake Motions. Two distinctly different time-history accelerograms were used as input earthquake excitations.
- Tunnel Embedment Depth. Most cut-and-cover tunnels are built at shallow depths. Various embedment depths were used to evaluate the effect of the embedment depth effect.

A total number of 36 dynamic finite element analyses were carried out to account for the variables discussed above. Based on the results of the analyses, a simplified procedure incorporating soil-structure interaction for the racking analysis of rectangular tunnels was developed. The step-by-step procedure is outlined below (Wang, 1993).

Step 1: Estimate the free-field ground strains $\gamma_{\max}$ (at the structure elevation) caused by the vertically propagating shear waves of the design earthquakes, see Section 13.5.1.1 in deriving the free-field ground strain using various methods. Determine $\Delta_{free\text{-}field}$, the differential free-field relative displacements corresponding to the top and the bottom elevations of the box structure (see Figure 13-20) by using the following expression:

$$\Delta_{free-field} = H \cdot \gamma_{\max} \qquad 13\text{-}20$$

Where:

H = height of the box structure

Alternatively site-specific site response analysis may be performed to provide a more accurate assessment of $\Delta_{free\text{-}field}$. Site-specific site response analysis is recommended for tunnels embedded in soft soils.

Step 2: Determine the racking stiffness, K_s, of the box structure from a structural frame analysis. The racking stiffness should be computed using the displacement of the roof subjected to a unit lateral force applied at the roof level, while the base of the structure is restrained against translation, but with the joints free to rotate. The ratio of the applied force to the resulting lateral displacement yields K_s. In performing the structural frame analysis, appropriate moment of inertia values, taking into account the potential development of cracked section, should be used.

Step 3: Determine the flexibility ratio, F_r, of the box structure using the following equation:

$$F_r = (G_m / K_s) \cdot (W/H) \qquad 13\text{-}21$$

Where:

W = Width of the box structure
H = Height of the box structure
G_m = Average strain-compatible shear modulus of the surrounding ground between the top and bottom elevation of the structure
K_s = Racking Stiffness of the box structure

The strain-compatible shear modulus can be derived from the strain-compatible effective shear wave velocity, C_{se}, see Equation 13-4).

Detailed derivation of the flexibility ratio, F_r, is given by Wang (1993).

Step 4: Based on the flexibility ratio obtained from Step 3 above, determine the racking coefficient, R_r, for the proposed structure. The racking coefficient, R_r, is the ratio of the racking distortion of the structure embedded in the soil, Δ_s, to that of the free-field soil, $\Delta_{free\text{-}field}$, over the height of the structure (see Figure 13-20):

$$R_r = \Delta_s / \Delta_{free\text{-}field} \qquad 13\text{-}22$$

From a series of dynamic finite element analyses, Wang (1993) presented results showing the relationship between the structure racking and the flexibility ratio, F_r. The values of R_r vs. F_r obtained from the dynamic finite element analyses are shown in Figure 13-21(a) and Figure 13-21(b). Also shown in these figures are curves from closed-form static solutions for circular tunnels (refer to Section 13.5.1.1). The solutions shown in the figures are from the full-slip solution presented by Wang (1993) and Penzien

(2000) and the no-slip solution presented by Penzien (2000). As can be seen in the figures, the curves from the closed-form solutions provide a good approximation of the finite element analysis results. These curves can therefore be used to provide a good estimate of the racking of a rectangular tunnel as a function of the flexibility ratio defined by Equation 13-21. The analytical expressions for the curves in Figure 13-21 are:

For no-slip interface condition:

$$R_r = \frac{4(1-\nu_m)F_r}{3-4\nu_m+F_r} \qquad \text{13-23}$$

For full-slip interface condition:

$$R_r = \frac{4(1-\nu_m)F_r}{2.5-3\nu_m+F_r} \qquad \text{13-24}$$

Several observations can be made from Figure 13-21. When F_r is equal to zero, the structure is perfectly rigid, no racking distortion is induced, and the structure moves as a rigid body during earthquake loading. When F_r is equal to 1, the racking distortion of the structure is approximately the same as that of the soil (exactly equal to that of the soil for the no-slip interface condition). For a structure that is flexible relative to the surrounding ground, ($F_r > 1$), racking distortion of the structure is greater than that of the free-field. As noted by Penzien (2000), if the structure has no stiffness (i.e., $F_r \rightarrow \infty$), R_r is approximately equal to $4(1-\nu_m)$, which is the case of an unlined cavity.

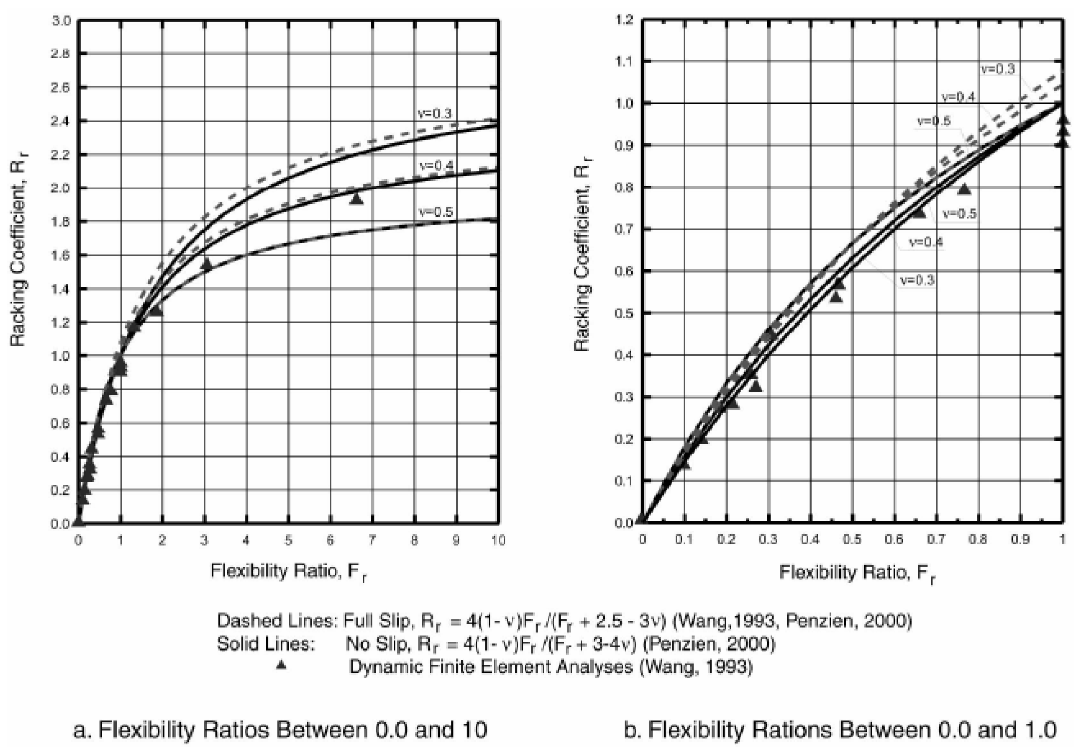


Figure 13-21 Racking Coefficient R_r for Rectangular Tunnels (MCEER-06-SP11, Modified from Wang, 1993, and Penzien, 2000)

Step 5: Determine the racking deformation of the structure, Δ_s, using the following relationship:

$$\Delta_s = R_r \cdot \Delta_{free-field} \qquad 13\text{-}25$$

Step 6: The seismic demand in terms of internal forces as well as material strains are calculated by imposing Δ_s upon the structure in a frame analysis as depicted in Figure 13-22 (MCEER-06-SP11). Results of the analysis can also be used to determine the detailing requirements.

As indicated in Figure 13-22, two pseudo-static lateral force models are recommended. The more critical responses from the two models should be used for design. If the displacements are large enough to cause inelastic deformation of the structure, inelastic soil-structure interaction analyses should be performed to assess structural behavior and ensure adequate strength and displacement capacity of the tunnel structure.

Under the loading from the design earthquake, inelastic deformation in the structure may be allowed depending on the performance criteria and provided that overall stability of the tunnel is maintained. Detailing of the structural members and joints should provide for adequate internal strength, and ductility and energy absorption capability if inelastic deformation is anticipated.

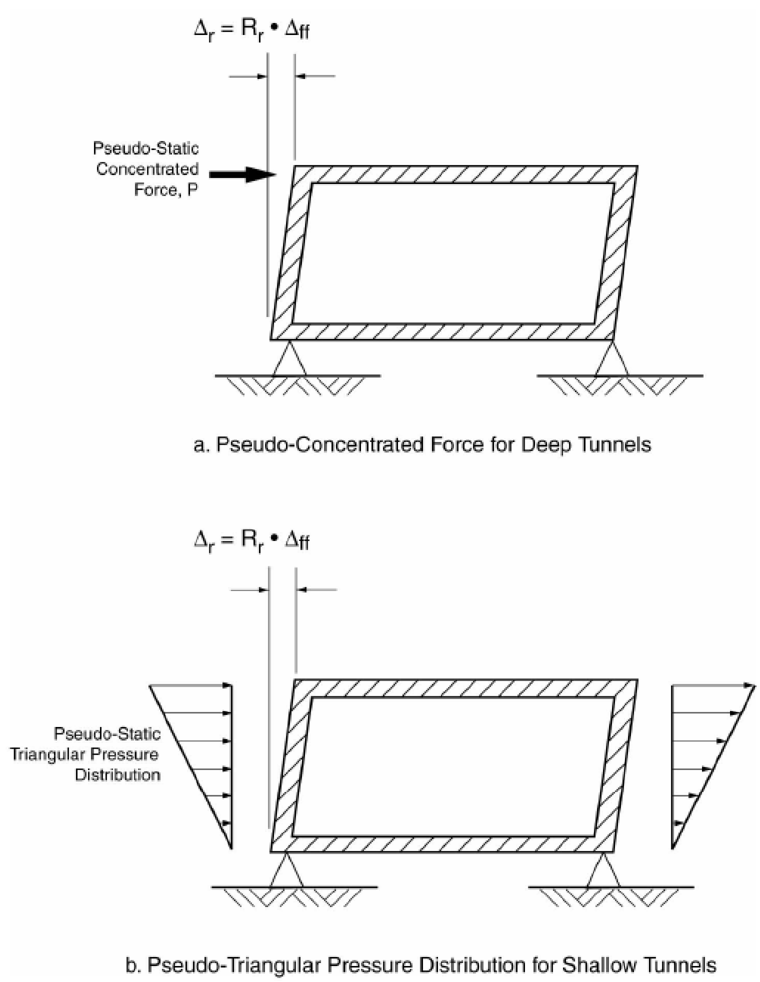


Figure 13-22 Simplified Racking Frame Analysis of a Rectangular Tunnel (MCEER-06-SP11, Modified from Wang, 1993)

Step 7: The effects of vertical seismic motions can be accounted for by applying a vertical pseudo-static loading, equivalent to the product of the vertical seismic coefficient and the combined dead and design overburden loads used in static design. The vertical seismic coefficient can be reasonably assumed to be two-thirds of the design peak horizontal acceleration divided by the gravity. This vertical pseudo-static loading should be applied by considering both up and down direction of motions, whichever results in a more critical load case should govern.

Step 8: Seismic demands due to racking deformations and vertical seismic motions are then combined with non-seismic loads using appropriate load combinations. A load factor of 1.0 is recommended in the load combination criteria.

13.5.1.4 Numerical Modeling Approach

The analytical solutions presented in Sections 13.5.1.2 and 13.5.1.3 for transverse response of tunnel structures (i.e., ovaling for circular tunnels and racking for rectangular tunnels) have been developed based on ideal conditions and assumptions as follows:

- The tunnel is of completely circular shape for ovaling response or rectangular shape for racking response.
- The material surrounding the tunnel is uniform and isotropic.
- The tunnel is very deep, away from the surface so that no reflection/refraction of seismic wave from the ground surface.
- Only one single tunnel is considered. There is no interaction from other tunnel(s) or structure(s) in proximity.

The actual soil-structure system encountered in the field for underground structures are more complex than the ideal conditions described above and may require the use of numerical methods. This is particularly true in cases where a very important tunnel structure is located in a severe seismic environment.

For transverse ovaling/racking analysis, two-dimensional finite element or finite difference continuum method of analysis is generally considered adequate numerical modeling approach. The model needs to be developed with the capability of capturing SSI effects as well as appropriate depth-variable representations of the earth medium and the associated free-field motions (or ground deformations) obtained from site-response analyses of representative soil profiles.

There are three types of two-dimensional continuum method of analysis that have been used in engineering practice and they are described in the following sections.

<u>Pseudo-Static Seismic Coefficient Deformation Method:</u> In pseudo-static seismic coefficient deformation method, the ground deformations are generated (induced) by seismic coefficients and distributed in the finite element/finite difference domain that is being analyzed. The seismic coefficients can be derived from a separate one-dimensional, free-field site response analysis.

The pseudo-static seismic coefficient deformation method is suitable for underground structures buried at shallow depths. The general procedure in using this method is outlined below:

- Perform one-dimensional free-field site response analysis (e.g., using SHAKE program). From the results of the analysis derive the maximum ground acceleration profile expressed as a function of depth from the ground surface.
- Develop the two-dimensional finite element (or finite difference) continuum model incorporating the entire excavation and soil-structure system, making sure the lateral extent of the domain (i.e., the horizontal distance to the side boundaries) is sufficiently far to avoid boundary effects. The geologic medium (e.g., soil) is modeled as continuum solid elements and the structure can be model either as continuum solid elements or frame elements. The side boundary conditions should be in such a manner that all horizontal displacements at the side boundaries are free to move and vertical displacements are prevented (i.e., fixed boundary condition in the vertical direction and free boundary condition in the horizontal direction). These side boundary conditions are considered adequate for a site with reasonably leveled ground surface subject to lateral shearing displacements due to horizontal excitations.
- The strain-compatible shear moduli of the soil strata computed from the one-dimensional site response analysis should be used in the two-dimensional continuum model.
- The maximum ground acceleration profile (expressed as a function of depth from the ground surface) derived from the one-dimensional site response analysis is applied to the entire soil-structure system in the horizontal direction in a pseudo-static manner.

- The analysis is executed with the tunnel structure in place using the prescribed horizontal maximum acceleration profile and the strain-compatible shear moduli in the soil mass. It should be noted that this pseudo-static seismic coefficient approach is not a dynamic analysis and therefore does not involve displacement, velocity, or acceleration histories. Instead, it imposes ground shearing displacements throughout the entire soil-structure system (i.e., the two-dimensional continuum model) by applying pseudo-static horizontal shearing stresses in the ground. The pseudo-static horizontal shearing stresses increase with depth and are computed by analysis as the product of the total soil overburden pressures (representing the soil mass) and the horizontal seismic coefficients. The seismic coefficients represent the peak horizontal acceleration profile derived from the one-dimensional free-field site response analysis. The lateral extent of the domain in the two-dimension analysis system should be sufficiently far to avoid boundary effects. In this manner, the displacement profiles at the two side boundaries are expected to be very similar to that derived from the one-dimensional free-field site response analysis. However, in the focus area near the tunnel construction the displacement distribution will be different from that of the free field, reflecting the effects of soil-structure interaction (i.e., presence of the tunnel structure) as well as the effect that portion of the earth mass is removed for constructing the tunnel (i.e., a void in the ground).

Pseudo-Dynamic Time-History Analysis The procedure employed in pseudo-dynamic analysis is similar to that for the pseudo-static seismic coefficient deformation method, except that the derivation of the ground displacements and the manner in which the displacements are imposed to the two dimension continuum system are different. The pseudo-dynamic analysis consists of stepping the soil-structure system *statically* through displacement time-history simulations of free-field displacements obtained by a site response analysis performed using vertically propagating shear waves (e.g., SHAKE analyses). Under the pseudo-dynamic loading, the transverse section of a tunnel structure will be subject to these induced ground distortions. Figure 13-23 shows an example of a two-dimensional continuum finite element analysis performed for an immersed tube tunnel structure subject to static stepping of a pseudo-dynamic displacement time history. In this model both the geologic medium (e.g., soil) and the tunnel structure were modeled as continuum solid elements. As indicated in the figure, in addition to the natural in-situ soils, the model can also consider the effect of the backfill material (within the dredged trench) on the ovaling/racking response of the tunnel structure. If warranted, the inelastic behavior of the tunnel structure can also be accounted for and incorporated into the model.

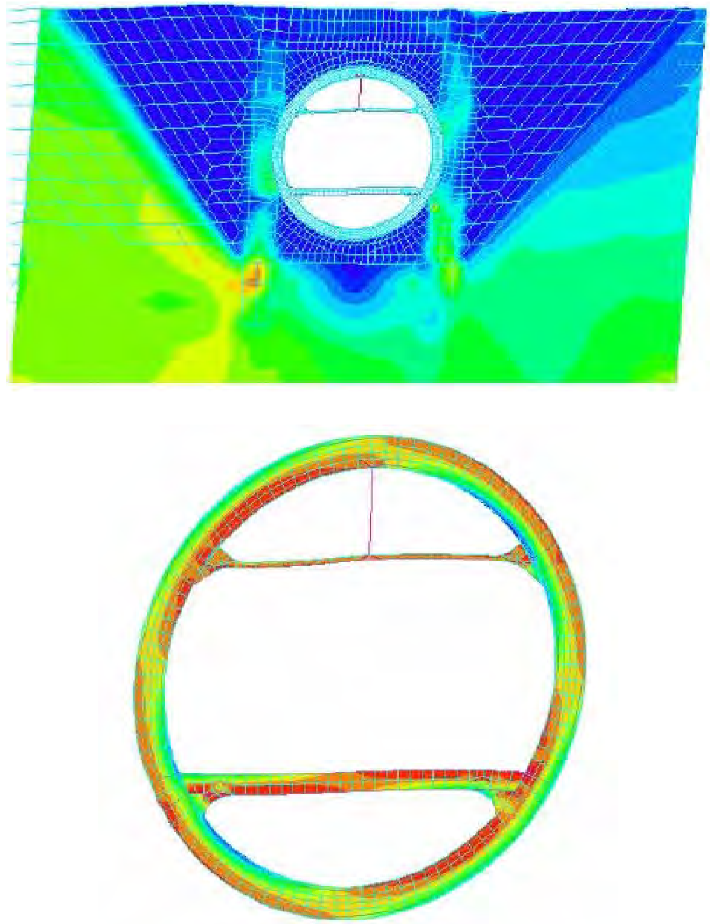


Figure 13-23 Example of Two-dimensional Continuum Finite Element Model in Pseudo-Dynamic Displacement Time-History Analysis

The model shown in Figure 13-23 includes both the geologic medium and the structure in one model. Alternatively, the analysis can also be performed in a *de-coupled* manner, where the tunnel structure is analyzed separately from the surrounding geologic medium. This *de-coupled* analysis involves the following two general steps:

- Computing the *scattered* ground displacements at the perimeter of the tunnel cavity subject to the design earthquake, without the tunnel structure (note that these are the *scattered* motions and not the *free-field* motions, due to the presence of the cavity in the ground). A two-dimensional site response analysis is generally performed using continuum finite element/difference plane-strain model to derive these scattered ground displacements. The soil (continuum) models and the associated properties shall be consistent with the soil strain levels that are expected to develop during the earthquake excitations (i.e., using strain level compatible soil properties).
- Impose the displacements obtained at the perimeter of the tunnel cavity onto the tunnel structure (e.g., a frame model) through interaction soil springs to evaluate the seismic response of the tunnel structure. When appropriate, the interface conditions between the tunnel frame and the surrounding soil should allow for the formation of gaps as well as slippage.

Dynamic Time History Analysis: Generally, the inertia of a tunnel is small compared to that of the surrounding geologic medium. Therefore, it is reasonable to perform the tunnel deformation analysis using pseudo-static or pseudo-dynamic analysis in which displacements or displacement time histories are statically applied to the soil-structure system. The dynamic time history analysis can be used to further refine the analysis when necessary, particularly when some portion(s) of the tunnel structure can respond dynamically under earthquake loading, i.e., in the case where the *inertial effect* of the tunnel structure is considered to be significant.

In a dynamic time history analysis, the entire soil-structure system is subject to *dynamic* excitations using ground motion time histories as input at the base of the soil-structure system. The ground motion time histories used for this purpose should be developed to match the target design response spectra and have characteristics that are representative of the seismic environment of the site and the site conditions (refer to Section 13.2.3).

Figure 13-24 shows a sample dynamic time history analysis using a two-dimensional continuum finite difference model for a cut-and-cover box structure. It should be noted in the figure that, the side boundary conditions in a dynamic time history analysis should be in such a manner that out-going seismic waves be allowed to pass through instead of being trapped within the soil-structure system being analyzed. Special *energy absorbing boundaries* should be incorporated into the model to allow radiation of the seismic energy rather than trapping it.

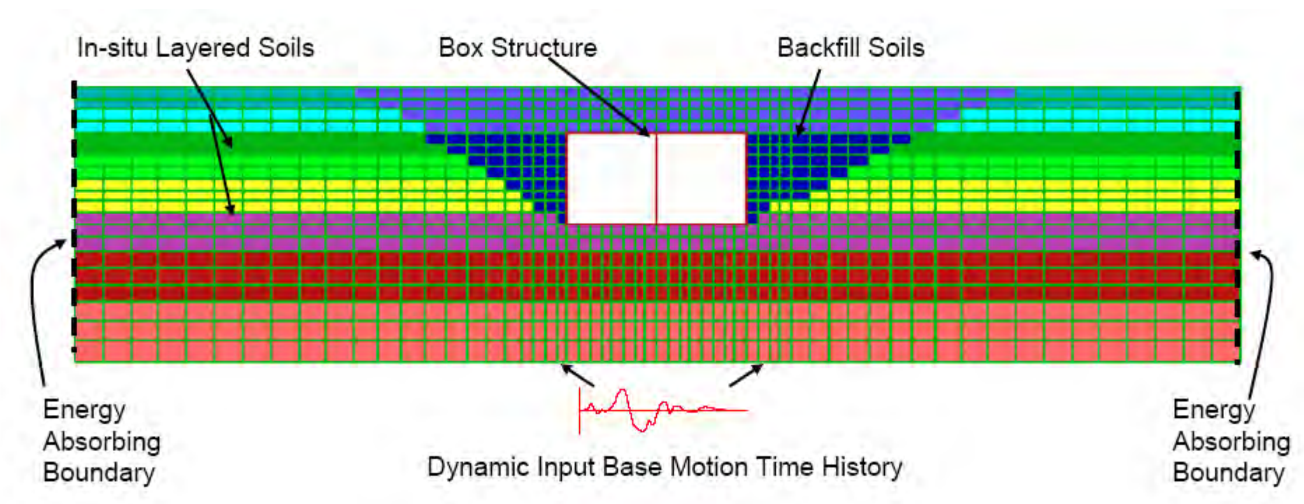


Figure 13-24 Sample Dynamic Time History Analysis Model

13.5.2 Evaluation of Longitudinal Response of Tunnel Structures

Similar to the procedures discussed for the evaluation of transverse response of tunnel structures, the evaluation procedures for the longitudinal response of tunnel structures can also be based on either simplified analytical method or more complex numerical modeling approach, depending on the degree of complexity of the soil-structure system, the seismic hazard level, and the importance of the structures. Section 13.5.2.1 discusses the simplified free-field deformation method, which ignores the soil-structure interaction effects. A refined method is then presented in Section 13.5.2.2 that considers the soil-structure interaction effects based on analytical beam-on-elastic-foundation theory. The more comprehensive and complex method using numerical modeling approach is discussed in Section 13.5.2.3.

13.5.2.1 Free-field Deformation Procedure

This procedure assumes that the tunnel lining conforms to the axial and curvature deformations of the ground in the free-field (i.e., without the presence of the tunnel). While conservative, this assumption provides a reasonable evaluation because, in most cases, the tunnel lining stiffness is considered relatively flexible to that of the ground. This procedure requires minimum input, making it useful as an initial design tool and as a method of design verification.

The lining will develop axial and bending strains to accommodate the axial and curvature deformations imposed by the surrounding ground. St. John and Zahran (1987) developed solutions for these strains due to compression P-waves, shear S-waves, and Rayleigh R-waves.

The strains ε due to combined axial and curvature deformations can be obtained by combining the longitudinal strains generated by axial and bending strains as follows:

For P-waves:

$$\varepsilon = \frac{V_P}{C_P}\cos^2\phi + Y\frac{A_P}{C_P^2}\sin\phi\cos^2\phi \qquad \text{13-26}$$

For S-waves:

$$\varepsilon = \frac{V_S}{C_S}\sin\phi\cos\phi + Y\frac{A_S}{C_S^2}\cos^3\phi \qquad \text{13-27}$$

For R-waves:

$$\varepsilon = \frac{V_R}{C_R}\cos^2\phi + Y\frac{A_R}{C_R^2}\sin\phi\cos^2\phi \qquad \text{13-28}$$

Where:

V_P = Peak particle velocity of P-waves at the tunnel location
V_S = Peak particle velocity of S-waves at the tunnel location
V_R = Peak particle velocity of R-waves at the tunnel location
A_P = Peak particle acceleration of P-waves at the tunnel location
A_S = Peak particle acceleration of S-waves at the tunnel location
A_R = Peak particle acceleration of R-waves at the tunnel location
C_P = Apparent propagation velocity of P-waves
C_S = Apparent propagation velocity of S-waves
C_R = Apparent propagation velocity of R-waves

Y = Distance from neutral axis of tunnel cross section to the lining extreme fiber
ϕ = Angle at which seismic waves propagate in the horizontal plane with respect to the tunnel axis

It should be noted that:

- S-waves generally cause the largest strains and are the governing wave type
- The angle of wave propagation, ϕ, should be the one that maximizes the combined axial strains.

The horizontal propagation S-wave velocity, C_S, in general, reflects the seismic shear wave propagation through the deeper rocks rather than that of the shallower soils where the tunnel is located. In general, this velocity value varies from about 2 to 4 km/sec. Similarly, the P-wave propagation velocities, C_P, generally vary between 4 and 8 km/sec. The designer should consult with experienced geologists/seismologists for determining C_S and C_P. In the absence of site-specific data, the horizontal propagation S-wave and P-wave velocities can be assumed to be 2.5 km/sec and 5 km/sec, respectively.

When the tunnel is located at a site underlain by deep deposits of soil sediments, the induced strains may be governed by the R-waves. In such deposits, detailed geological/seismological analyses should be performed to derive a reliable estimate of the apparent R-wave propagation velocity, C_R.

The combined strains calculated from Equations 13-26, 13-27, and 13-28 represent the seismic loading effect only. To evaluate the adequacy of the structure under the seismic loading condition, the seismic loading component has to be added to the static loading components using appropriated loading combination criteria developed for the structures. The resulting combined strains are then compared against the allowable strain limits, which should be developed based on the performance goal established for the structures (e.g., the required service level and acceptable damage level).

13.5.2.2 Procedure Accounting for Soil-Structure Interaction Effects

If a very stiff tunnel is embedded in a soft soil deposit, significant soil-structure interaction effects exist, and the free-field deformation procedure presented above may lead to an overly conservative design. In this case, a simplified beam-on-elastic-foundation procedure should be used to account for the soil-structure interaction effects. According to St. John and Zahran (1987), the effects of soil-structure interaction can be accounted for by applying reduction factors to the free-field axial strains and the free-field curvature strains, as follows:

For axial strains:

$$R = 1 + \frac{E_l A_l}{K_a}\left(\frac{2\pi}{L}\right)^2 \cos^2\phi \qquad \text{13-29}$$

For bending strains:

$$R = 1 + \frac{E_l I_l}{K_h}\left(\frac{2\pi}{L}\right)^4 \cos^4 \phi \qquad 13\text{-}30$$

Where:

- E_l = Young's modulus of tunnel lining
- A_l = Cross sectional area of the lining
- K_h = Transverse soil spring constant
- K_a = Longitudinal soil spring constant
- L = Wave length of the P-, S-, or R-waves
- I_l = Moment of inertia of the lining cross section.

It should be noted that the axial strain calculated using the procedure presented above should not exceed the value that could be developed using the maximum frictional forces, $Q_{\max}$, between the lining and the surrounding soils. $Q_{\max}$ can be estimated using the following expression:

$$Q_{\max} = \frac{fL}{4} \qquad 13\text{-}31$$

Where:

- f = Maximum frictional force per unit length of the tunnel

13.5.2.3 Numerical Modeling Approach

Numerical modeling approach for the evaluation of longitudinal response of a tunnel structure is desirable for cases where tunnels encounter abrupt changes in structural stiffness or run through highly variable subsurface conditions (where the effect of spatially varying ground motions due to local site effect becomes significant). These conditions include, but are not limited to, the following:

- When a regular tunnel section is connected to a station end wall or a rigid, massive structure such as a ventilation building.
- At the junctions of two tunnels or at the tunnel/cross-passage interface.
- When a tunnel traverses two distinct geological media with sharp contrast in stiffness, for example, a tunnel passing through a soil/rock interface.
- When a tunnel is locally restrained from movements by any means (i.e., "hard spots").

Numerical analysis for the evaluation of longitudinal response of a tunnel structure is typically performed by a three-dimensional pseudo-dynamic time history analysis in order to capture the two primary modes of deformation: axial compression/extension and curvature deformations. As discussed previously, since the inertia of a tunnel is small compared to that of the surrounding geologic medium, the analysis is generally performed by using the pseudo-dynamic approach in which *free-field* displacement time histories are statically applied to soil springs connected to the model of the tunnel (to account for the soil-structure interaction effect). The general procedure for the pseudo-dynamic time history analysis in the longitudinal direction involves the following steps.

- The free-field deformations of the ground at the tunnel elevation are first determined by performing dynamic site-response analyses. For the longitudinal analysis, the three-dimensional effects of ground motions as well as the local site effect including its spatially varying effect along the tunnel alignment should be considered. The effect of wave travelling/phase shift should also be included in the analysis.
- Based on results from the site response analyses, the free-field ground displacement time histories are developed along the tunnel axis. The free-field displacement time histories at each point along the tunnel axis can be defined at the mid-height and mid-width of the tunnel, can be further defined in terms of three time-history displacements representing ground motions in the longitudinal, transverse and vertical directions.
- A three-dimensional finite element/difference structural model is then developed along the tunnel axis. In this model, the tunnel is discretized spatially along the tunnel axis, while the surrounding soil/ground is represented by discrete springs. If inelastic structural behaviour is expected, non-linear inelastic structural elements should be used to represent the tunnel structure in the model. Similar to the ground motions, the soil/ground springs are also developed in the longitudinal, transverse horizontal and transverse vertical directions. The properties of the springs shall be consistent with those used in the site response analysis in described above. If non-linear, the behaviour of the soil/ground should be reflected in the springs. As a minimum, the ultimate frictional (drag) resistance (i.e., the maximum frictional force) between the tunnel and the surrounding soil/ground should be accounted for in deriving the longitudinal springs to allow slippage mechanism, should it occur.
- The computed design displacement time-histories described above are then applied, in a statically stepping manner, at the support ends of the soil/ground springs to represent the soil-tunnel interaction. The resulting sectional forces and displacements in the structural elements (as well as in the tunnel joints if applicable) are the seismic demands under the axial/curvature deformation effect.

13.6 SEISMIC EVALUATION PROCEDURES - GROUND FAILURE EFFECTS

As mentioned earlier, the greatest risk to tunnel structures is the potential for large ground movements as a result of unstable ground conditions (e.g., liquefaction and landslides) or fault displacements. In general, it is not feasible to design a tunnel structure to withstand large ground displacements. The proper design measures in dealing with the unstable ground conditions may consist of:

- Ground stabilization
- Removal and replacement of the problem soils
- Re-route or deep burial to bypass the problem zone

With regard to the fault displacements, the best strategy is to avoid any potential crossing of active faults. If this is not possible, then the general design philosophy is to accept and accommodate the displacements by either employing an oversized excavation, perhaps backfilled with compressible/collapsible material, or using ductile lining to minimize the instability potential of the lining. In cases where the magnitude of the fault displacement is limited or the width of the sheared fault zone is considerable such that the displacement is dissipated gradually over a distance, design of a strong lining to resist the displacement may be technically feasible. The structures, however, may be subject to large axial, shear and bending forces. Many factors need to be considered in the evaluation, including the stiffness of the lining and the ground, the angle of the fault plane intersecting the tunnel, the width of the fault, the magnitude as well as orientation of the fault movement. Analytical procedures are generally used for evaluating the effects of fault displacement on lining response. Some of these procedures were originally developed for buried pipelines (ASCE Committee on Gas and Liquid Fuel Lifelines, 1984). Continuum finite-element or finite-difference methods have also been used effectively for evaluating the tunnel-ground-faulting interaction effects.

The following sections will discuss briefly the general considerations and methodology used in dealing with various types of ground failure effects.

13.6.1 Evaluation for Fault Rupture

General: Assessing the behavior of a tunnel that may be subject to the direct shear displacements along a fault includes, first, characterizing the free-field fault displacement (i.e., displacements in the absence of the tunnel) where the fault zone crosses the tunnel and, second, evaluating the effects of the characterized displacements on the tunnel.

Figure 13-25 is an example of such a relationship, which shows that the amount of displacement is strongly dependent on earthquake magnitude and can reach maximum values of several feet or even tens of feet for large-magnitude earthquakes.

Analyzing Tunnels for Fault Displacement: When subjected to fault differential displacements, a buried structure with shear and bending stiffness tends to resist the deformed configuration of the fault offset, which induces axial and shear forces and bending moments in the structure. The axial deformation is resisted by the frictional forces that develop at the soil-tunnel interface in the axial direction, while shear and curvature deformations are caused by the soil resistance normal to the tunnel lining or walls.

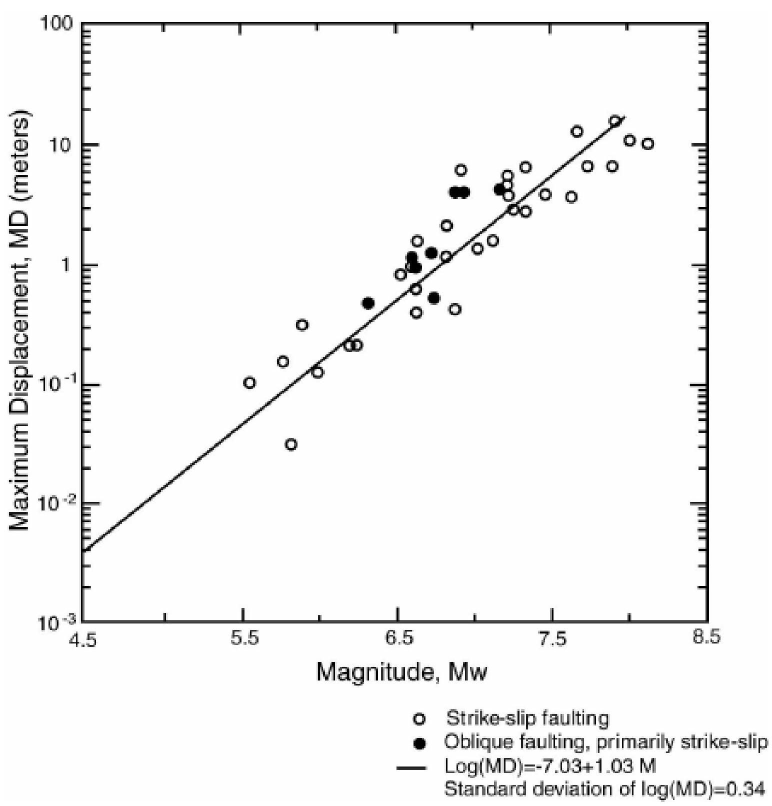


Figure 13-25 Maximum Surface Fault Displacement vs. Earthquake Moment Magnitude, M_w (Wells and Coppersmith, 1994)

In general, analytical procedures for evaluating tunnels subjected to fault displacements can follow those used for buried pipelines. Three analytical methods have been utilized in the evaluation and design of linear buried structures (ASCE Committee on Gas and Liquid Fuel Lifelines, 1984). They are: (1) Newmark-Hall procedure, (2) Kennedy et al. procedure, and (3) Finite element approach. For detailed evaluation of transportation tunnels at fault crossing, however, it is generally believed that finite element method is more appropriate than other methods. The finite element method is preferred because it can incorporate realistic models of the tunnel and surrounding geologic media. The tunnel is modeled using finite elements, which may incorporate nonlinear behavior (Figure 13-26).

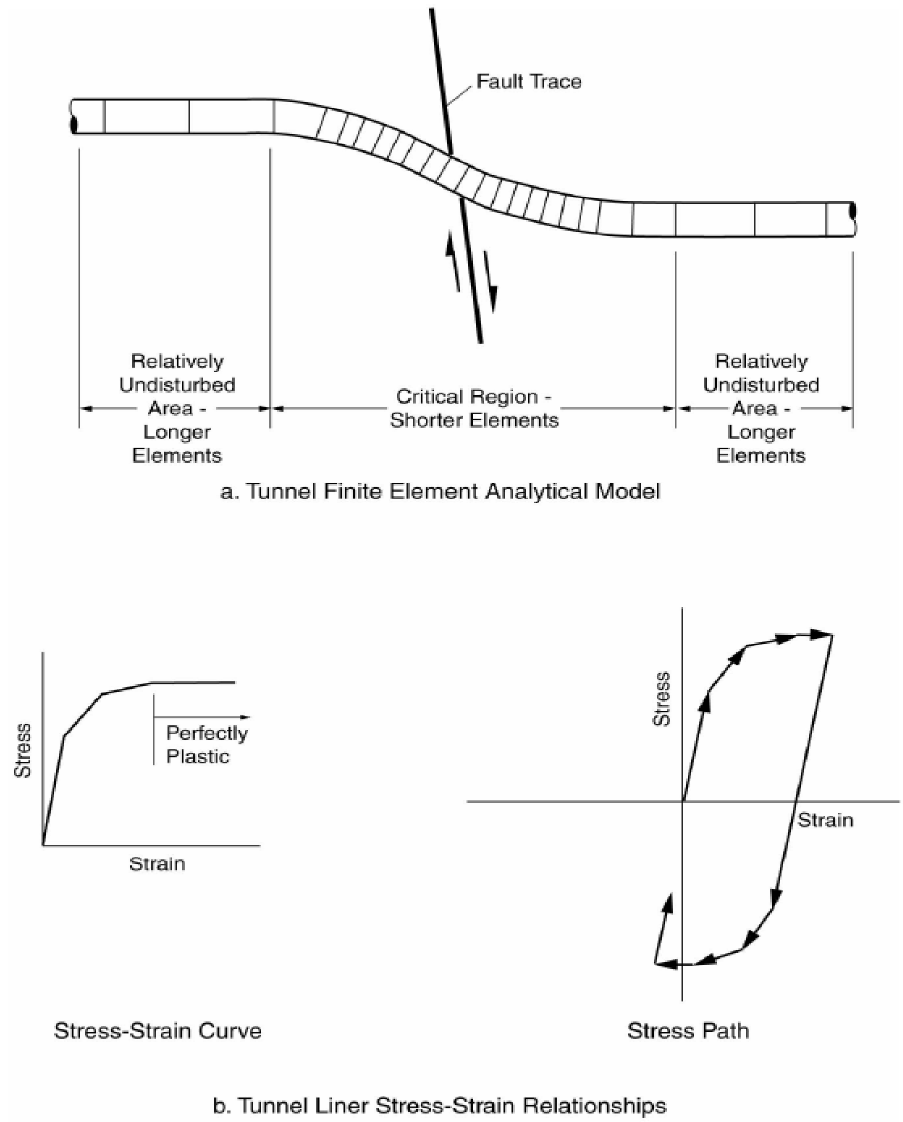


Figure 13-26 Analytical Model of Tunnel at Fault Crossing (ASCE, 1984)

Transverse and axial springs connected to the tunnel model soil normal pressures on the tunnel lining or walls and axial frictional resistance (Figure 13-27); these springs may also incorporate nonlinear behavior if applicable (Figure 13-28). Many commercially available finite element codes may be considered for analyzing the response of tunnels to fault displacement.

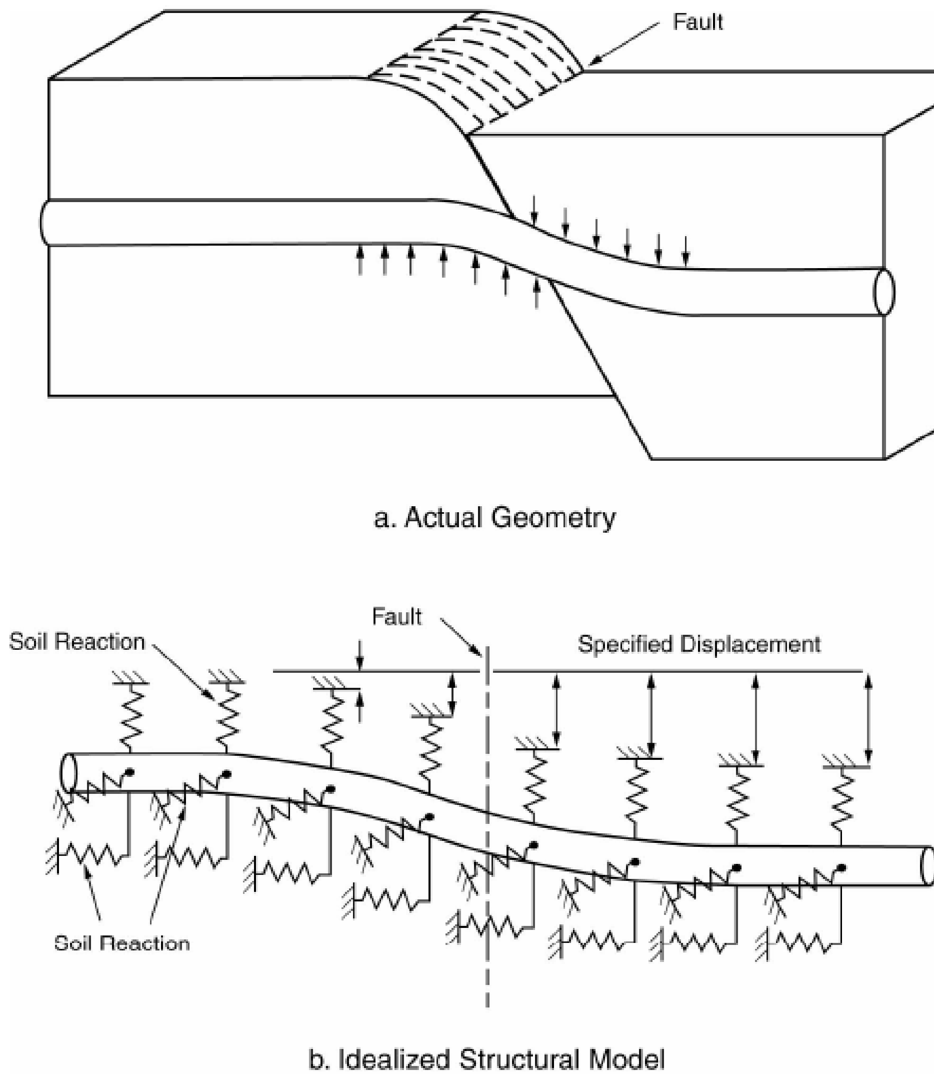


Figure 13-27 Tunnel-Ground Interaction Model at Fault Crossing (ASCE Committee on Gas and Liquid Fuel Lifelines, 1984)

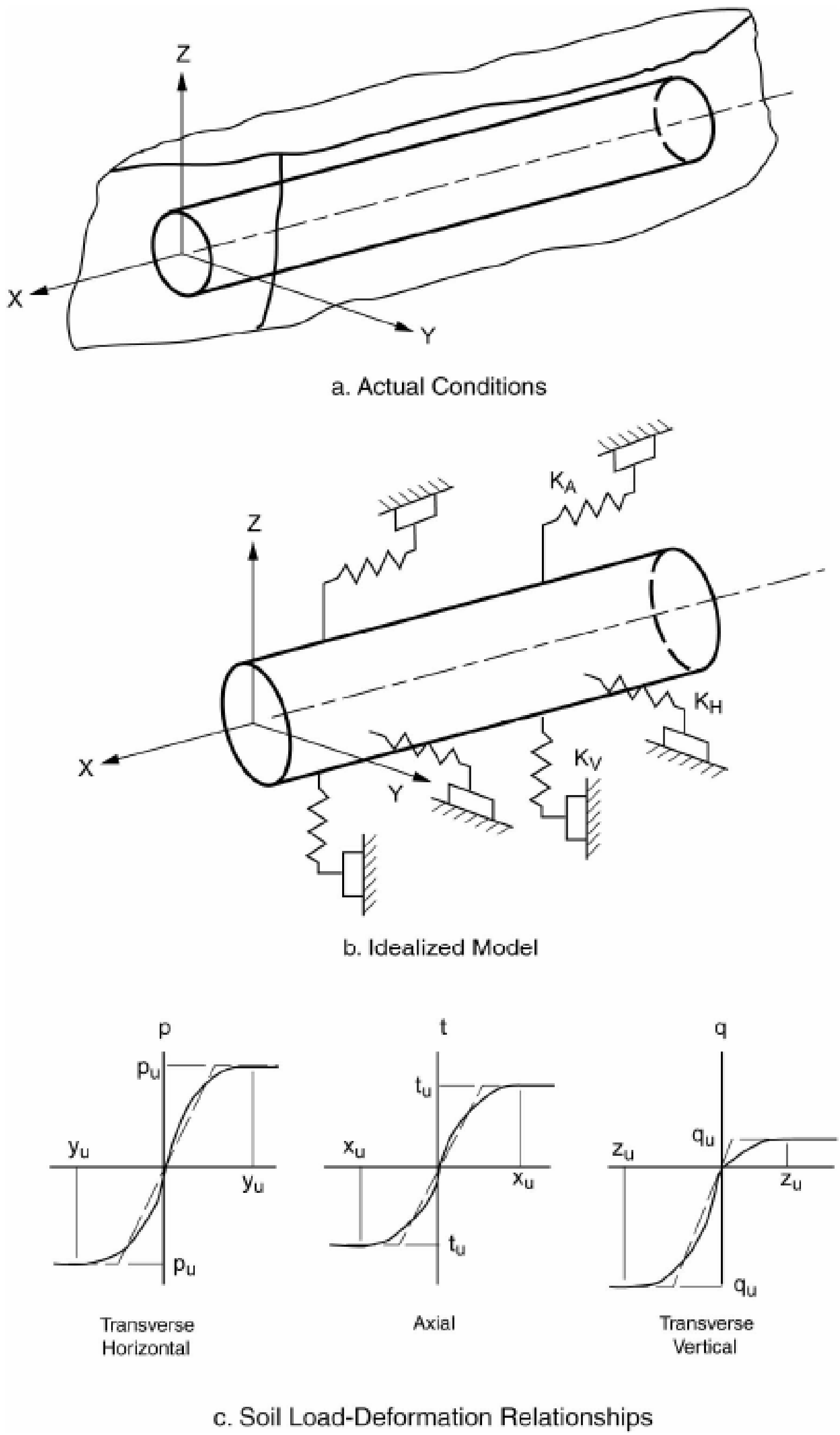


Figure 13-28 Analytical Model of Ground Restraint for Tunnel at Fault Crossing (ASCE Committee on Gas and Liquid Fuel Lifelines, 1984)

13.6.2 Evaluation for Landsliding or Liquefaction

If liquefiable soil deposits or unstable soil masses susceptible to landsliding are identified along the tunnel alignment, then more detailed evaluations may be required to assess whether liquefaction or landsliding would be expected to occur during the design earthquake and to assess impacts on the tunnel.

If slope movements due to landsliding or lateral spreading movements due to liquefaction intersect a tunnel, the potential effects of these movements on the tunnel are similar to those of fault displacement. As is the case for fault displacements, tunnels generally would not be able to resist landsliding or lateral spreading concentrated displacements larger than a few inches without experiencing locally severe damage.

If liquefaction were predicted to occur adjacent to a tunnel lining or wall, a potential consequence could be yielding of the lining or wall due to the increased lateral earth pressure in the liquefied zone. The pressure exerted by a liquefied soil may be as large as the total overburden pressure. The potential for liquefaction to cause uplift of a tunnel embedded in liquefied soil, or for the tunnel to settle into the soil, should also be checked.

CHAPTER 14
TUNNEL CONSTRUCTION ENGINEERING

14.1 INTRODUCTION

This chapter focuses mostly on mined/bored tunnel construction engineering; the engineering that must go into a road tunnel project to make it constructible. Each decision made during the planning (Chapter 1) and design of a road tunnel project has impacts on the constructability, cost and schedule of the Work. This chapter will look at these cost drivers and how they influence the project's final cost. Planning, design and finally construction operations should be guided by people experienced in the actual construction of these underground works so that the projects are constructible. The schedules must be realistic and reflect all the restrictions that are imposed on the project whether they are physical, political or third party. Cost estimates must reflect the actual schedule time needed to complete the work and account for all the restrictions imposed on the project.

Tunneling is unique when compared to other types of civil construction. In non tunnel projects like a large building or treatment plant there are usually many places to work at the same time, so the work can continue even if there is a problem holding up work at one location. Tunnels are long linear undertakings with few opportunities to perform the work at more than one location. Tunnels are also a series of repetitive operations each of which usually must be finished before the next can be started.

This uniqueness and the linear, repetitive nature of the work must be understood by the planners and builders of tunnel projects to control and manage the project to a successful conclusion.

Perhaps the most significant factor impacting tunnel cost and schedule is the type of geologic material that the tunnel will be mined through and the amount of ground and surface water that will be encountered or crossed. Tunnels are mined through rock, soil or a combination of both. The geology encountered determines the tunneling methods that will be used, the speed that the tunnel can be constructed and the types of specialized equipment that are required.

The geologic material can also present some unique health and safety concerns that must be accounted for in the planning and construction of underground projects. Gas, petroleum, contamination, voids in the ground, hot water or large quantities of groundwater all pose safety concerns that must be addresses so that the workers building the tunnels are provided an environment free of hazards.

Of similar importance to the tunneling methods and hours of operation are the communities that the tunnel will pass under, the locations of the major work shafts or portals from which the work will be serviced and the streets through which the equipment, personnel and material will get to and from the worksite as well as how the muck removed from the tunnel is disposed of.

All of these factors will have impacts on the cost and schedule of underground projects and in fact represent risks to the project. These risks must be acknowledged, allocated and mitigated. Dealing with these risks can be accomplished through the contractual language between the parties to a tunnel project, or if not dealt with or if dealt with inappropriately, contractor claims or lawsuits.

14.2 CONSTRUCTABILITY

The design for an underground project must be constructible. Too often road tunnels are designed by competent engineers who have never actually built anything. Their designs minimize the volume of excavation and concrete but are difficult to build. Underground construction is expensive due to the large proportion of labor used during the construction, the high wages paid to these workers and the linear nature of the work. In order for our tunnels to be less expensive to build, designers must also be schooled in how tunnels are built so they can recognize that their decisions on size, shape, location and esthetics all have cost impacts.

A brief discussion of the labor portion of the cost of underground construction is in order so that designers can start to understand how their decisions impact these costs. Most underground civil construction is performed in a union environment. The union provides skilled labor that performs specific job functions. Typically there is a crew actually performing the work. This crew will consist of miners, miner foremen, operators to run and maintain the equipment, electricians to maintain the power that runs the equipment and provides the necessary lighting levels as well as supervisory people. These folks actually performing the repetitive operations are called the heading or direct labor crews. These crews are supported by an entire separate group of people that supply the project with needed power, material, transportation, maintenance and overall project management. These are called the service crews. The service crew can be as big as the direct labor crews. If you have 25+/- direct labor doing the work you also have 25+/- people supporting the work. These two or more crews are being paid whether the work is going forward or not.

One typical example of where the design of a tunnel project can impact the cost is in a location where a tunnel must be widened out to accommodate an exit or entrance or even an emergency pull-off. In most designs you will see a constantly changing cross section going from the road tunnel and widening out to accommodate the exit, entrance or emergency parking area. This looks nice, is visually pleasing and minimizes both the excavated volume and the amount of concrete that is required in the lining, but is it easy to build and what does it add to the cost?

Most contractors will come back to the project's owner and propose to accommodate the same structure in a stepped fashion instead of a smooth transition. Why? It is relatively easy to excavate the transition cross sections in a rock tunnel (more difficult in a soft ground tunnel operation) and certainly a smoothly transitioning excavation does minimize the volume of material that is taken out. However the lining operation becomes real tricky and costly.

The smooth transition requires different custom built forms for each foot of the structure. There is no, or limited, reuse of forms and most importantly each of these custom forms must be built in place, used and removed thus slowing down the lining operation. Each use of a custom form requires both the direct crew and the service crew to be used for a longer duration driving up the cost and increasing the schedule for the whole project.

Now look at what the use a larger cross section or a stepped transition can do for the cost and schedule. If we simply go from the typical tunnel size to the full size required for the exit, entrance or parking lane we pay for some extra excavation and concrete but we only now have two forms (one extra) to build use and remove. If using just two different cross sections is not possible, then a multi-stepped transition can help to minimize the time and money spent building, using and removing all the specialized forms. An evaluation must be made whether it is faster and less costly to remove extra material and place extra concrete or to install, use and remove all the specialized forms.

So how do we make our designs more constructible? One way is to include a construction expert on the design team. This construction expert would then sit with the designers reviewing what approach they want to utilize, make suggestions on how the design could be more easily built, make sure that all the site constraints have been addressed and providing insight into how a contractor would price the designs so that modifications of the design can be made to control cost and schedule.

14.3 CONSTRUCTION STAGING AND SEQUENCING

14.3.1 Construction Staging

Each underground project is unique; however, there are certain requirements and functions common to most or all tunnels. Each project requires one or several places from which the work can be prosecuted. All projects require large quantities of labor, material and equipment be brought underground to excavate and support the tunnel and large quantities of muck and ground water must be removed from the tunnel. All projects therefore require land area to set up the contractor's offices, shops storage yards, muck storage piles, electrical substations and many other space needs. It therefore is logical that the more space that can be made available to the contractor to locate needed structures, store needed materials and allow for the movement of materials and equipment into and out of the worksite, the more efficient and less costly the operation will be. On the other hand the smaller the available worksite the more expensive and less efficient the operation will be (Figure 14-1).

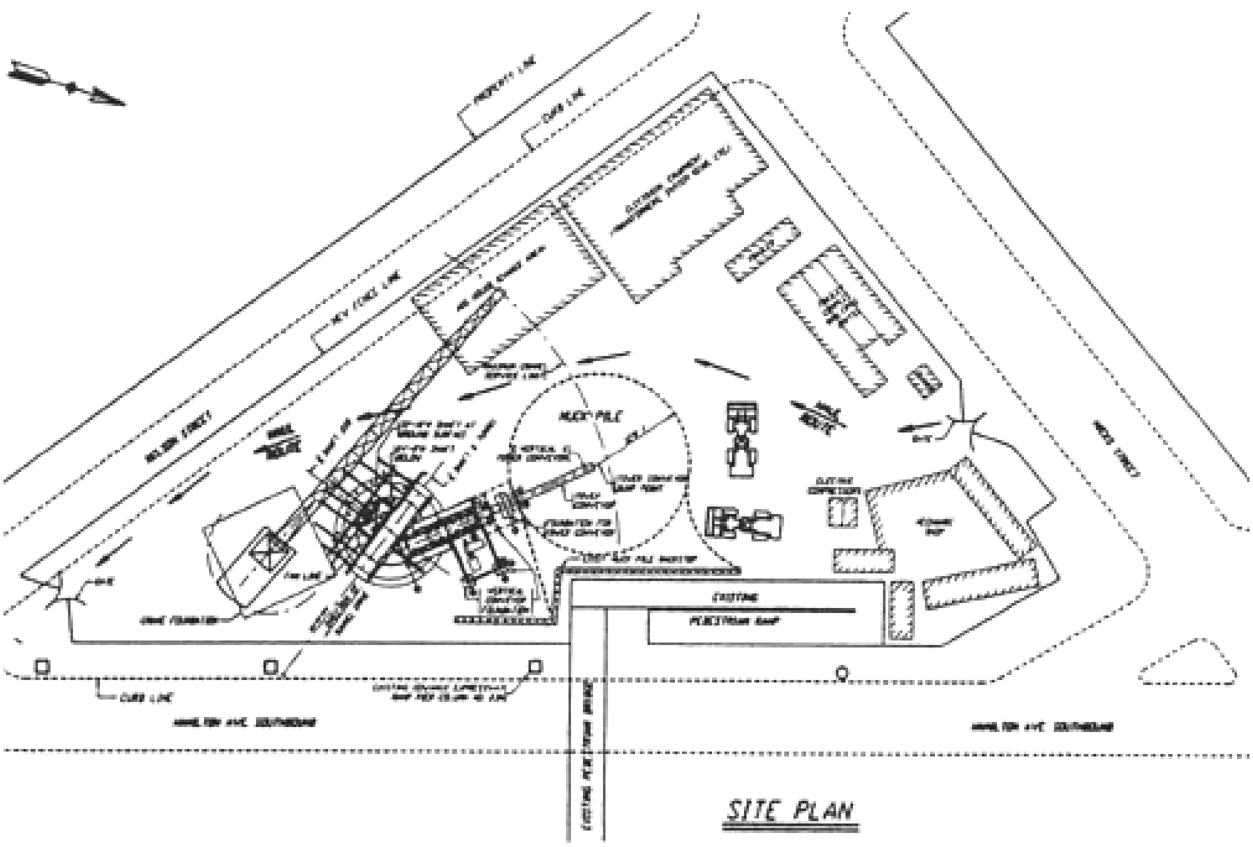


Figure 14-1 Confined Worksite and Staging Area

Underground projects serviced by shaft(s) require room to excavate the shaft. There should be room all around the shaft to allow equipment access and easy flow of the work around the shaft location. Typically owners, who must acquire property to locate the shaft will minimize the size of the property and thereby minimize their expenditure for property acquisition. This can be shortsighted. Paying more for more room can actually provide for a more efficient operation, lowering the overall cost for the work and providing the owner the opportunity to sell off the extra property after the project is completed at a higher price thereby further lowering the total cost of the construction.

Portal projects benefit from not having the expense and schedule impact of excavating and supporting the shaft(s) but also require property on one or both sides of the project to enable the contractor to efficiently prosecute the work (Figure 14-2). Portal areas for a road tunnel may be limited by existing geotechnical hazards.

Figure 14-2 Tunnel Portal

14.3.2 Construction Sequencing

Underground construction is a series of individual activities that must be completed before the subsequent activities can start. This series of unique activities is then repeated and repeated until the operation is complete. For tunnels that employ drilling and blasting to create the tunnel opening the series is, “drill, load, shoot, muck and support.” Each round is drilled a certain length or depth using a pre-engineered drill pattern. Once the drilling is done the explosives are loaded into the drill holes and “wired up”. The equipment and crews are then pulled back a safe distance from the loaded face and the blast is “shot”. Exhaust gasses produced by the explosives are removed from the face and fresh air is sent to the heading area. After around 30 minutes the crew is brought back into the area to scale or knock down any loose rock and remove the excavated material or “muck”. Once the muck is removed, the initial tunnel support is installed to make the excavated opening stable and safe for the crew to work under. The cycle is complete and the tunnel has been advanced some distance. The next round can be started when all of these activities have been completed.

In TBM excavated tunnels there is also a defined sequence of activities needed to advance the heading. The TBM usually completes this series much faster that in drill & blast tunnels but the elements remain similar. The TBM cuts into the rock or earth a certain distance at the same time the muck is removed by conveyor to either waiting muck cars or to a continuous horizontal conveyor, so the TBM is able to combine these two operations thereby saving time and speeding up the tunnel progress. After the end of the TBM's stroke (the hydraulic pistons used to push the TBM cutting head into the rock have a defined length) the excavation is stopped and the TBM readied to start the next excavation cycle. While this is happening the length of tunnel that has just been exposed most be supported to provide a stable and safe opening. The TBM can sometimes be configured to perform this support function concurrently with the excavation sequence depending on the size of the tunnel opening, the type of ground being excavated and the design of the machine. This can be another advantage of using a TBM but does not change the fact that this operation must be done before the next excavation cycle can begin.

Tunnels are usually stabilized for long term use by placing an internal final concrete liner. The concrete lining operation also contains a series of individual steps that must be completed in sequence before the next length of tunnel can be lined.

14.4 MUCKING AND DISPOSAL

"Muck" is the industry term for excavated material produced during the advancement of the tunnel. All tunnel mining produces muck. This excavated material must be removed from the working face of the tunnel so that the next advance can be made. Tunneling is a series of individual steps, each of which must be completed before the next can start. Once the muck is produced it must be removed from the tunnel and finally disposed of in a legal manner or used as fill for some portion of the tunnel project or other project where it could have a beneficial use.

Muck is actually a broken down state of the insitu material through which the tunnel is driven. Because the natural material is disturbed by either blasting, cutting with a TBM, roadheader or cut out with a bucket excavator the volume of muck removed actually is larger than the natural bank material. This swell is usually approximated as 70% to 100% more in rock and 25% to 40% for soil.

The material that is excavated must be removed from the tunnel. The method chosen to remove this material depends on many factors such as the diameter or size of the excavation, the length of the tunnel excavated from any given heading, the material being moved, the grade of the tunnel being driven and whether the material is going to a shaft for removal or a portal. Horizontal conveyor belts are commonly used for large excavated tunnels that are longer than a few thousand feet and are excavated by a TBM (Figure 14-3). Conveyors can move a large quantity of material quickly. Conveyors require that the excavated material be of relatively uniform small size so that it will sit in the belt during the transfer to the shaft or portal. Conveyors can sometimes be used with a drill and blast excavation method if the contractor employs a crusher to make the drill and blast rock a more even and smaller consistency. This crushing is necessary to ensure that the material sits nicely on the belt, is small enough that when it is loaded onto the belt it does not damage or rip the belt material. Conveyors are usually limited to a grade (or slope) less than 18 degrees to successfully transport muck, but this is never an issue in road tunnels. Conveyors can transport rock or soil. The soil must not be too wet or it will not transport well. Conveyors can also be used in tunnels where there are curves in the alignment but this requires some special care and equipment.

Figure 14-3 Horizontal Muck Conveyor

Material that is too wet to carry on a conveyor belt can sometimes be pumped out of the tunnel through a pipeline from the TBM to the shaft or portal. This method is successfully used on soft ground tunnels where the material is clay like or where sufficient water (and often, conditioners) is mixed with the excavated material to make it slurry like.

For smaller tunnels excavated by a TBM, contractors often choose to load the excavated muck into rail cars and haul it out of the tunnel using locomotives. Rail haulage also has some limitations such as the grades are usually limited to less than 4%, a great amount of rolling stock is required and great care must be paid to maintaining the track.

Once the muck arrives at the shaft or portal it must be off loaded and then disposed of. Figure 14-4 shows a muck train dumping at a tunnel portal. A shaft is a vertical hole through which all excavated material must be lifted and removed and through which all material required for the tunneling operations must be lowered to the tunnel level. In addition all personnel working on or inspecting the tunnel must come in and out of this shaft. In other words it is a busy place. There are many ways to transport the muck up the shaft. Muck cars can be lifted one by one up the shaft, dumped in a pile on the surface and lowered back down to the tunnel. Muck cars can be dumped into a hopper at the bottom of the shaft and then loaded into a bucket that is hoisted to the top and dumped or the muck from the hopper could be loaded onto a vertical conveyor and conveyed to the top of the shaft and dumped onto a pile or hopper. Similarly the muck can be pumped to the surface and deposited on a horizontal conveyor, a stockpile or run through a processing plant to remove the water and the residual dumped on a pile or into hoppers.

Figure 14-4 Muck Train Dumping at Portal

Portals provide easier access to a tunnel since they eliminate the bottleneck that the shaft imposes. Muck is easier to remove at a portal since track can be paced on the ground or on an elevated trestle so that muck cars can be pulled outside to dump their loads onto a muck pile.

The really important thing to remember is that tunneling is a series of steps that must be done and complete before the cycle can start again. This means that any disruption in the muck removal operation will delay the start of the next round or the next advance. If you cannot get rid of the muck you can not produce more! This is also true once the muck reaches the surface. There should be a place to store the muck that is brought out of the tunnel until it can be loaded into trucks or rail cars and hauled away. Without this storage capability on the surface (Figure 14-5), all muck brought out of the tunnel must immediately be loaded into surface trucks or rail cars for disposal. If there is a holdup in the surface trucking or rail cars then no more muck can be brought out and the tunnel advance must stop. This situation is called being "muck bound" and must be avoided at all costs. The more muck storage that is available the more unlikely it will be for a project to become muck bound. Work sites must be large enough to provide this storage cushion, the larger a worksite the bigger the cushion. It is increasingly more difficult to find available land in and around cities to provide a suitably large worksite. Typically urban sites are small and therefore special care must be taken to ensure a steady stream of vehicles to remove the muck as it is produced, and to deliver workers and materials as needed. Thought must also be given to the hours of operation allowed in urban tunnel projects. If the hours of operation for surface work are restricted, i.e., surface work is not allowed after 10 PM at night, then in order to operate the tunnel 24 hours per day, there must be some place to store muck underground that is produced on the shift where no surface work is allowed, and construction noises must be kept below a threshold based on local ordinances and/or certain realistic decibel levels.

Figure 14-5 Surface Muck Storage Area

14.5 HEALTH & SAFETY

Construction Engineering and safety go hand in hand. Underground construction is inherently a dangerous undertaking. Work goes on in a noisy environment, in close quarters often with moving heavy machinery. Careful attention must be paid to the layout of the worksites; workers must be protected at all times. The overriding philosophy must be that, “everyone goes home safely at the end of their shift”.

Every step of the operation should be planned with safety in mind. The normal surface safety concerns are also appropriate for underground construction. Workers must be safeguarded from falling off of the work platforms used in the mining process. Workers must be protected from being struck by the moving equipment used throughout the mining process. Workers most be protected from being electrocuted. However there are also many additional hazards that workers must be protected from and guarded against.

Work underground involves mining through rock or soil or a combination of both. In order to excavate the opening required for the tunnel the natural properties of the ground are disturbed. The ground is usually not a homogeneous mass but has been subjected to massive forces of nature and has been altered. Once the opening has been excavated it must be supported in order for the workers to be protected from falling material, collapse or other deterioration of the tunnel roof or crown. So it is the job of the Construction Engineer to plan on making the tunnel opening stable to allow workers to move freely and without concern of falling material.

Because tunnels are by definition below the surface, lighting of the workspace is an important part of underground safety. OSHA has regulations governing all elements of working underground and the Construction Engineer must be familiar with them all. There are required levels of lighting for the actual work locations as well as the previously excavated openings. It is important to remember that the tunnels are long linear work places. As the tunnels are advanced more and more safety plant must be added along with the productive support elements.

One of the more challenging aspects of tunnel safety is the fact that workers must be constantly supplied with high quality breathable air. Again OSHA is very specific in its requirements. Each person underground must be supplied with 200 cubic feet per minute (cfm) of air. In addition much of the equipment underground is powered by internal combustion engines. Diesel fuel is the only fuel allowed underground. OSHA again has specific requirements for the equipment and for the amount of air that must be delivered to the underground for each and every piece of diesel equipment working underground. This diesel air requirement is in addition to the requirement for each and every person underground. The quality of the tunnel atmosphere must be tested on a regular schedule to ensure that sufficient quantities of oxygen are present and that concentrations of undesirable gasses and byproducts of the internal combustion engines are controlled to acceptable levels.

Also tests must be performed on a regular basin to ensure that the air movement across the excavated cross section is no less that 30 foot per minute.

If this were not enough, as discussed in Chapter 8, Mother Nature can often provide challenges to the safety of workers underground. There can be gasses underground that can seep into the tunnel opening after the excavation operation. These gasses can be poisonous like hydrogen sulfide or explosive like methane. Whenever these gasses are present or suspected to be present the Construction Engineer has additional OSHA requirements to be aware of and to follow. Extra ventilation will be required, in addition to the air needed for both people and diesel equipment and the required quantity can be substantial. Whenever these gasses are suspected there are extra requirements for continuous monitoring of the atmosphere with automatic shutdown of equipment should the gasses be detected in concentrations higher than allowed.

Water entering the tunnel opening is also a safety issue in tunnels. Most tunnels are excavated below the water table. The tunnel opening acts like a big drain and any water running through the rock or contained in the soil tends to collect in the tunnel. Water running through the tunnel bottom or invert can cause several potential safety issues. Tunnels can be accessed by one or more shafts, by a combination of shafts and portal or from a portal alone. It is desirable to drive tunnels up hill so that any water that seeps into the excavated opening flows away from the working face by gravity. This water is usually allowed to run in a ditch located at the side of the tunnel invert. Care must be taken that workers do not step into or fall into this ditch. The higher the inflow of water into the tunnel the greater the problem of safely conveying it back along the tunnel and finally out the shaft or portal.

Tunnels that are driven down hill have the problem that water flows to and accumulates at the working face. This collected water must be removed from the work area by pumping. The water is pumped through a pipe at the side of the excavation. This pipe must extend all the way to the shaft or portal where it can be removed from the tunnel. Water can also enter the tunnel in sudden large flows. These can be very dangerous occurrences and any tunnel where this is a possibility extra care must be taken in the planning for worker safety. Tunnels under bodies of water are of particular concern for this risk of sudden large inflows of water.

Fires in tunnels are especially dangerous and can lead to extensive damage and risk to worker's safety and life. The Tunnel Construction Engineer must be aware of this potential danger and plan to mitigate the risk at every stage of the project. Most tunnels are driven from one point to another from a single point of entry. This single point of entry is what makes tunnel fires so dangerous and concerning as shown in (Figure 14-6). The tunnel environment contains numerous potential sources of fire. Equipment can malfunction and catch fire. Workers using welding or burning torches can set off a fire. Leaking hydraulic fluid or fuel from equipment can be ignited by a stray spark or discarded cigarette. Conveyor belts used to transport muck can build up heat from rubbing on or over something and ignite. All these possible fire risks, and more must be addressed by the Construction Engineer to minimize the possibility of a fire or to minimize the potential damage and injuries resulting from a fire. Only retardant material and hydraulic fluid should be allowed on any underground equipment or material. Fire suppression systems should be required for all underground equipment, conveyor belt motors and storage magazines. Vertical muck removal belts should be equipped with deluge water systems to dump large quantities of water on any belt fire event. Fire and life safety issues during operation and maintenance of road tunnels are not included in the scope of this Manual.

Figure 14-6 Fire in Work Shaft

Of equal importance in dealing with tunnel fires is how to best provide for the safety of the workers underground. This can be accomplished in several ways. Rescue chambers, where workers can take refuge in a fire, are fully equipped and supplied with independent air supplies and insulation can be deployed along the tunnel as the tunnel is advanced. Equally important the tunnel can be planned with intermediate access points that can be fully equipped to be able to remove workers from the tunnel when the tunnel has been excavated past these locations.

The Tunnel Construction Engineer must also be certain to make sure that the job specifications require strict compliance with all safety measures and regulations local, state and national. The Engineer must stress to the designer and the owner that money spent on worker and job site safety is money well spent since the cost of accidents and replacing structures damaged or destroyed by a fire event is so high.

14.6 COST DRIVERS AND ELEMENTS

There are numerous cost drivers associated with underground construction. These can be grouped into physical, economic and political.

14.6.1 Physical Costs

The single most important driver of project cost is the ground through which the tunnel will be driven. The ground controls the methods and equipment used to drive the tunnel, the support elements that will be needed to ensure that the excavated cross section remains stable and safe for the personnel constructing the tunnel and the final lining needed for long term stability of the structure. In addition the ground through which the tunnel is driven will contain varying amounts of ground water that will dictate the pumping requirements, waterproofing needs and lining quality that will ensure a dry tunnel environment.

The use that the tunnel will serve also has a significant impact on the costs. Tunnels for roads and rail must be dry to safeguard the traveling public so a watertight structure is imperative. Road and rail tunnels are also grade restrictive and curvature restricted which also impact project cost. Tunnels that will service as road and rail infrastructure must be able to deliver large quantities of fresh air throughout the length of the tunnel and be able to remove smoke and heat developed during a fire incident anywhere in the tunnel. Large ventilation structures or in line fan systems are needed to supply this air and remove the smoke.

In rail or road tunnels refuge areas or rest areas are often needed along with on and off ramps or connections to outside rail or road systems.

14.6.2 Economic Costs

All tunnels require personnel, equipment, materials incorporated into the physical structure, materials that are consumed during the construction of the tunnel along with insurances, bonds, offices, shops and other indirect elements. These all impact the cost of the project. The largest portion of these costs is the actual cost of labor. Labor is broken down into the labor actually driving the tunnel or the direct or heading crews; the support crews that provide all the needed supplies of the tunnel, maintain the equipment used during the tunnel driving operations and provide access to and from the tunneling operation and the supervision needed to ensure that all the components work together in the required sequence.

Material is another major cost component of tunnel operations, Materials like cement, steel, copper wiring are all very price volatile now due to strong worldwide demand. Currently the price escalation of key materials is a significant cost driver and one that is often not addresses in the contract specifications as a separate cost. Tunnels require large quantities of both permanent and consumable materials in a constant stream.

We have also the continual cost of disposing of the muck or excavated material that is produced during the tunnel operations. Muck can sometimes be sold off by the contractor or owner to help reduce the cost of tunnel construction. However the market for this material is not guaranteed and often the contractor must pay to haul this muck away and also pay to dispose of it at approved dump locations. More and more regulations governing the disposition of materials are driving up the cost of tunnel construction.

Bonds and Insurance are smaller components of tunnel costs that are becoming cost drivers due to the increased scrutiny being imposed by the insurance and surety industry. Since most owners require both bonds and insurance on their projects by law and as risk management tools any contractor that cannot qualify for bonds and insurance cannot bid the project. After the terrorists attacks of September 11 and some high profile corporate failures, the marketplace for both bonds and insurance has tightened up and many providers have actually stopped writing bonds and certain types of insurances.

14.6.3 Political Costs

Significant costs are placed on projects by either the communities through which the tunnels will be mined or by the owner agencies by the requirements and restrictions incorporated into the specifications. Tunnels are expensive undertakings even without these restrictions but when concessions to various groups are added to the requirements the costs can skyrocket. Tunnels built in rural areas experience few of these political costs but those driven through urban settings can experience significant costs due to these restrictions. Typical restrictions are, mandating certain types of construction to minimize community disruptions, i.e., mining an underground cavern instead of digging down from the surface or not having a work shaft at a certain location because it is too close to neighbors. Restrictions on the hours worked is commonly employed when the tunnel is in a urban location. Tunnels are a cyclical series of operations where one cannot start till the predecessor is complete. With restrictions on the hours of operation fewer steps can be completed in the reduced time so the job takes longer. In one case an owner agency allowed 24 hour tunneling (recognizing that this is a typical mode of operation) but limited the hours that could be worked at the surface where the muck is brought out to be trucked away. In order to compensate for this reduced time the underground opening had to be made larger, so that the muck that was produced during the time where no surface work could be done, could be stored underground awaiting that time of day when it could be brought to the surface and trucked away, the political cost of being a good neighbor.

Owners might drive up the cost of doing underground work by restricting what costs are recoverable by the contractor in a change order or claim situation and by preventing the contractor from recovering delay costs if the delay is caused by the acts or inaction by the owner. These “No damage for delay” clauses might suggest to the contractors to incorporate into their bids these potential costs and the owner pays for them whether they occur or not.

14.7 SCHEDULE

The importance of the development and use of a realistic schedule and cost estimate for all phases of a project cannot be overemphasized. It is critical to understand the relationships among all the activities and costs that go into a project as well as the needs and interests of all those who are affected by the planning, design, construction, testing and commissioning of the work. With this understanding, projects can go forth in an orderly, predictable manner, which in the end benefits everyone.

The schedule is the road map of how the project progresses through all the necessary steps. It is advised that a comprehensive schedule be developed during the early stages of the conception of a project. During this early stage the project may be too immature to support realistic time durations but some time must be assigned to each and every component; such as planning, siteing, environmental process, permitting, right of way acquisition, preliminary and final design, bidding, contract award, construction, testing, commissioning start up and any activity or phase that is important to or has a cost for the project Owner. As the project develops and more of the actual scope and restrictions are known the schedule must be reevaluated and updated to reflect this new knowledge. The schedule development should be a living process that is used and revised constantly to be of maximum benefit to the project.

The realistic time needed to accomplish all aspects of the project must finally be reflected in the schedule. It makes no sense to handicap the tool (schedule) or the process by introducing artificial or incorrect restrictions or by putting unrealistic expectations into the schedule. In fact, these restrictions and incorrect assumptions always create problems later on in the project, usually in the form of delays, claims and higher costs. There can be a positive case made for an Owner to actually build some float time into the schedule, if possible, so that there is some way to cushion the effects of unknown occurrences that could impact the project schedule.

Unrealistic schedules sometime might result from external forces such as the desire to have a project completed in time for an upcoming event or election. These external forces always need to be acknowledged and addressed on a case-by-case basis. They can wreak havoc on a schedule, but they must be taken seriously. It should be noted that throughout a project's life, its schedule will be at the mercy of these external forces. Having said this, the best (and only) way to begin a project is with a realistic, well-thought out schedule and cost estimate. This will reduce the risk that the Owner Agency will be called on to defend a low-ball cost assumption and an inaccurate timeline necessary to complete the project. It is important to remember that the cost and schedule numbers that are initially released to the public are the ones that you will have to live with and defend throughout the project's life. It is much easier if these costs and schedules are reasonable and defendable, backed by professional experience and industry standards.

Numerous examples can be found where projects suffered from low cost and schedule pronouncements that were never achieved. In contrast, where realistic cost and schedules were developed, the Owner Agency managed the projects and was not constantly defending the numbers or the timeline. Having realistic schedules and budgets produces a "win-win" situation for both the Owner agency as well as the contractors by eliminating or at least minimizing the conflicts and finger pointing that can occur on a project that is squeezed for time and/or cash!

As the schedule of how the project is planned and built is developed, a timetable for the work also emerges. The schedule divides the work into discrete activities each with an amount of time needed for completion. Each activity is quantified with the important items of work such as linear feet of tunnel or cubic yards of concrete. Production rates are then applied to these activities and quantities. These discrete activities can then be combined in sequences that depict the way the designer anticipates the work to be constructed. These sequences can be linear or overlapping; but in the end, we have a roadmap of all the elements of the project, how they fit together and how long the project is expected to take.

Each of these discrete activities and the project as a whole are used to calculate the cost of doing the work. In the early stages of a project, these costs can be based on historical costs for similar size projects, in similar geologic conditions and in similar locations. These approximations of costs are useful for developing a potential cost for the work but, and these initial costs must not be used to develop an actual estimate.

The schedule is now the **roadmap** for developing the actual cost for the work. The Design Engineer should follow the procedures used by Contractors when they prepare their estimate for the bidding of the project. Typically, a contractor develops a crew of workers for each activity on the schedule. This crew is based on the work practices in the area, such as health and safety rules, where the project is located. The staffing is determined by the actual work to be accomplished, based on the local labor staffing requirements. After the crews are established the contractor will determine the productivity of the crew to accomplish the quantity of work associated with the activity This will determine the time required to do the work; or if the time is fixed, a determination is made as to how many workers are needed to perform the required quantity of work in the required time. To this labor, the contractor will add the equipment needed, the materials incorporated into the work and the materials consumed during the performance of the work.

This method is called a “bottom-up” estimate where all the components are established for each activity of work; then all these activities are combined into the total direct cost for the work. To this direct cost is added the indirect or costs not associated with any specific activity but needed for the overall construction of the project such as insurance, bonds, non union labor and costs of running the project and home offices.

By using a bottom-up estimate prepared by an estimator with some construction or contractor background, the Engineer’s Estimate will be more accurate and will better reflect the true costs for the work. This is the goal.

So why is a realistic schedule important? There are several reasons. The schedule gives the Owner an expectation of when the project is to be completed and ready for use. The schedule is used to coordinate the interfaces with other construction contracts within the project or external to the project, equipment procurement contracts and other interfaces. The schedule is also used to determine the cash flow and financing requirements, such as bond sales.

A schedule is used as the basis to determine the cost of the work. Labor makes up close to 30% of the cost of a tunnel estimate, so an accurate picture of the length of time that labor will be used on the project is important to the total cost the Owner, Contractor and Public will eventually have to pay.

There is an additional benefit that comes from using a realistic schedule as the basis of the engineer’s cost estimate. Once this is done then the schedule and estimate can be used to determine the magnitude of any claim proposed by the contractor (based on the contractor’s schedule and compared to the costs and schedule impacts claimed by the contractor) for delays or the impacts to the budget of Owner initiated extra work.

There are different levels of cost estimates. In early stages of a tunnel project, often a decision is made that for budget level or order of magnitude estimates, a bottoms up estimate is not necessary or appropriate since the project definition is not far enough along. Instead, a quick estimate can rely on unit price methods such as $-inch foot of tunnels in similar ground conditions. However, once an unrealistic number is estimated, it often stays with the project and establishes unrealistic expectation through out the life of the project as discussed previously. The sooner an experienced construction based scheduler and estimator gets involved the better the schedule and cost numbers will be, even if the estimator needs to make assumptions on typical design details.

14.8 CLAIMS AVOIDANCE AND DISPUTES RESOLUTION

Uncertainly and change in site condition on underground projects often leads to disputes, change orders and claims. Owners usually have years to plan a project, perform geotechnical investigations needed to understand the ground through which the tunnel will be built, and deal with all the regulatory agencies and third party abutters. Contractors are in business to make money. They usually have no input to the project plans, specifications, schedule or contracts but must accept these as given and in the space of a few months come up with a cost to perform the work and beat out all other contractors bidding the work. Underground projects are expensive, linear, and sequential, so any delay to the project leads to extra expense that the contractor will look to recover from the owner.

Recognizing the uncertain nature of underground construction and the need to make the contracts fairer, the federal government has mandated the use of a differing site condition clause in underground projects. This clause says in effect, that if the ground conditions differ from what was predicted or from what reasonably could have been anticipated in similar work then the owner would recognize this as additional costs and the contractor would be issued a change order to cover a portion of this extra cost and schedule. The alternative would be for the contractor to include into its bid a contingency to cover the potential costs if an unknown or unusual event occurred. If the contractor does this and the event does not occur then the owner is stuck paying for this uncertainty. The other option the contractor has is to not include any costs for these potential occurrences but to sue the owner to recover any additional cost should a risk event occur.

How can claims be avoided? One way is to incorporate a change condition clause into the contract. This is one indication that the owner is willing to share the risk on the project. Risk should be given to the party to the contract that is in the best position to control the risk. More and more owners are recognizing that they own the risk of the underground.

Another indication of the owner's stance on risk sharing to a contractor bidding the work is how the contract is worded in areas like, time related impacts of delays caused by the owner of outside agencies or third parties. Contracts that indicate that there will be "No Damage for Delay" make too plain to the contractor that the owner is not willing to share risk but is actively looking to transfer to the contractor all risks that they are not legally required to retain.

14.8.1 Disputes Resolution

Since disputes are inevitable in underground construction: how should they be dealt with? Suffering with these same issues the practitioners of underground construction got together and in 1974 produced a manual dealing with, "Better Contracting Practices for Underground Construction". This publication contained 14 recommendations to improve the way underground projects were managed. One of these recommendations was the use of a Disputes Review Board (DRB) and the use of Escrow Bid Documents.

A Dispute Review Board is usually a trio of underground experts experienced in the design and construction of underground projects that are brought together by both the owner and contractor to, on a regular basis, become familiar with the project, its progress and problems and to offer their opinion about who is right and wrong in any disputes that arise on the project that cannot be settled by the contracted parties. These "three wise men" as they are sometime referred to, must be impartial and have such standing in the underground in industry that their decisions are accepted.

In any dispute that the DRB is asked to weigh in on, both sides are allowed to lay out their positions and refute the positions of the other side. The DRB is allowed to ask questions and evaluate the "evidence" supplied by both sides. Usually the DRB issues a written decision that then is used as the basis of settling the dispute. One of the side benefits of using a DRB is that often contractors will work hard to reach a settlement with the owner instead of going to the DRB and in fact the presence of a DRB will prevent a contractor presenting frivolous or questionable issues to the DRB so as not to look bad to their peers.

Escrow Bid Documents was another recommendation in the Better Contracting Practices publication. An owner will require that all bidders submit with their bids or the low 3 bidders submit within several days of submitting their bids, all the documents, quotes and other information that the bidders used to produce their bids. These documents usually must conform to minimum formats and are sealed. The owner and the low bidder then open the sealed documents to ensure that all the required information is present and if not the additional info is then added. The complete documents are then sealed and stored with an independent agent. The documents are then available if there is a dispute and can be opened in the presence of both owner and contractor, to determine what was and was not included by the bidder in the cost at the time of the bid. After the project is complete the Escrow Bid Documents are returned to the contractor.

There are other methods of dispute resolution used to help settle issues that arise on underground projects, arbitration and mediation to mention a few.

14.9 RISK MANAGEMENT

By its nature, risk sometimes defies definition, and the most onerous risks are those that were not anticipated by designer, contractor, owner or by anyone else. A well structured risk management process will anticipate, to the extent possible, the potential risks, weigh their probability and effects, and plan for handling the risk to the degree necessary to de-risk the project through every phase from conception to completion. The project owner who does not use risk management often fails to control the cost, schedule, quality and safety of the work.

The origins of risk in tunneling and underground construction often stem from unanticipated obstructions, natural or manmade, soil and groundwater conditions differing from those anticipated; ground behavior differing from that ordinarily expected; and misinterpretation of ground conditions leading to the choice of inappropriate construction methods or equipment. Analysis of historical records, photos and maps, as well as a comprehensive geotechnical investigation plan and other exploratory work, help determine the ground conditions along the tunnel horizon and location of existing or abandoned structures along a tunnel alignment, thereby reducing risk. Administrative risks (e.g., site unavailability for external reasons) are as important to eliminate. Interface risks between adjacent contracts, including items such as potential for late delivery of site or facility by one contractor for use by another, are another type of risk that can derail a construction schedule. Underlying mitigation for risks on tunneling projects include design of features that reduce or eliminate the identified risk; selection of tunnel alignments that, where possible, avoid adverse ground conditions or avoid above ground sensitive structures; specification of minimum requirements for methods of tunneling and shaft construction coupled with monitoring and controls to be implemented during construction that identify adverse trends and warn against impending risks.

Risk assessment, risk analysis and risk management are required to assure the project is kept on schedule and within budget, and to provide greater accuracy in the application of project contingency. A comprehensive risk management process includes the use of risk workshops, development of an “actionable” risk register, risk analysis and the development of risk management and action plans. What’s important is early identification and communication of potential risk factors that might create delays and bottlenecks, followed by proactive management of threats to cost and schedule adherence and to identify opportunities for improvement (as shown in Figure 14-7).

Typically risk management starts by an owner and design engineer conduct a risk workshop in which all participants are encouraged to write down any and all events that could happen on and to the project and that could have impacts on the cost, schedule, quality, viability and/or safety of the project. In addition the participants need to try to determine the owner’s risk tolerance. What is insignificant, tolerable and intolerable to the owner for each of the major drivers of the Project? The Owner’s risk tolerances must be categorized on some scale so that they can be compared and weighed against cost drivers. On the schedule is 1 day delay acceptable? Is a week or a month tolerable? Is several months intolerable? The same for costs, depending on the size of the project, is $5M tolerable? Is $50 M intolerable? A scale or matrix (Figure 14-8) must be developed that rates risks consequences from inconsequential all the way to unacceptable so that choices can be made as to which to ignore, which to watch, and which to deal with or eliminate. These matrixes can be a 3x3, 5x5 or even 10x10. The more categories contained in the matrix the more effort is needed to manage this technical phase of the risk management process.

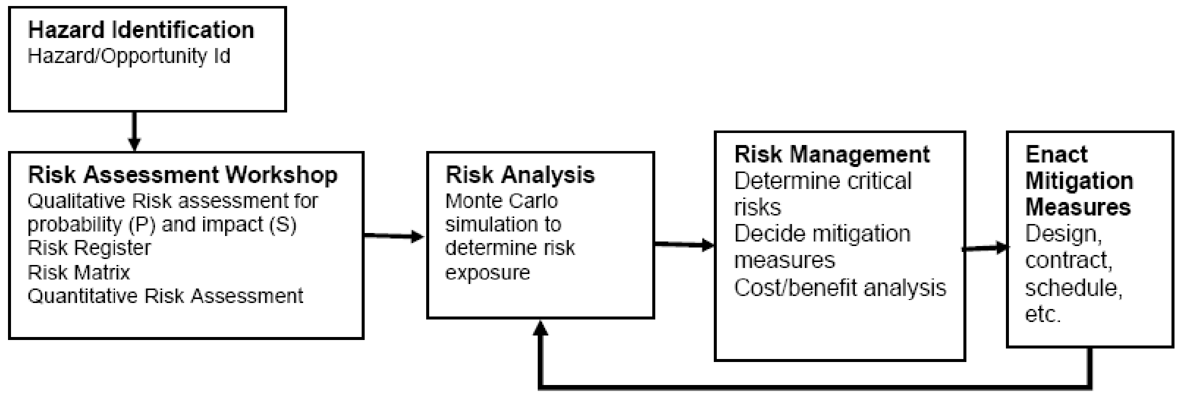


Figure 14-7 Risk Management Process

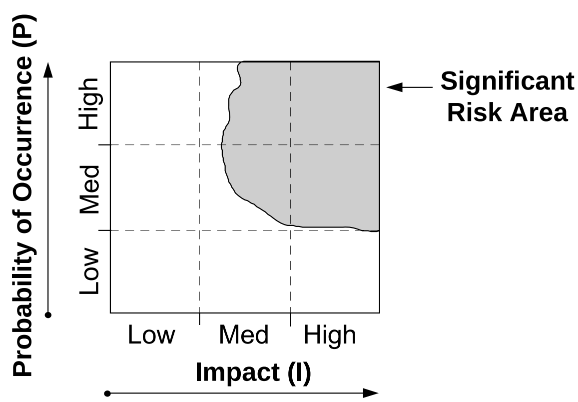


Figure 14-8 Typical Project Risk Matrix

A risk register is used as a way to catalogue the events that might happen on the project, and the probability and consequences if they occur. In addition it is also a tool to compare the risks, catalogue and mitigation measures chosen by the project team to either lessen the probability that the events occur or to lessen the consequences should they occur. The register also allows the project to keep track of all the mitigation efforts and the residual risks that remain. Knowing these residual risks allows an owner to then decide what to do with these residual risks. Residual risks can be accepted by the owner, passed on to the contractor, given to the insurance or bonding companies or can be candidates for additional mitigation. Once these events are catalogued then the workshop participants are asked to identify the probability that these events actually happening and if they happen, what would be the consequences or impacts on the project's cost, schedule, quality, viability and safety. Risk is actually the possibility of an event happening times the consequences that occur should the event happen.

The risk management process forms the basis of design development, accurate cost estimates and development of confident construction schedules. Risk Management and Action plans are developed based on the residual exposure after the anticipated reduction of the risks have been achieved. Costs can then be attributed to the mitigation of these risks. However, the process does not stop there. Through each phase of the project identified risks should be further evaluated in terms of ultimate risk exposure in schedule uncertainty, monetary value, probability and mitigation costs. Figure 14-9 illustrates the risk management process throughout phases of a project cycle.

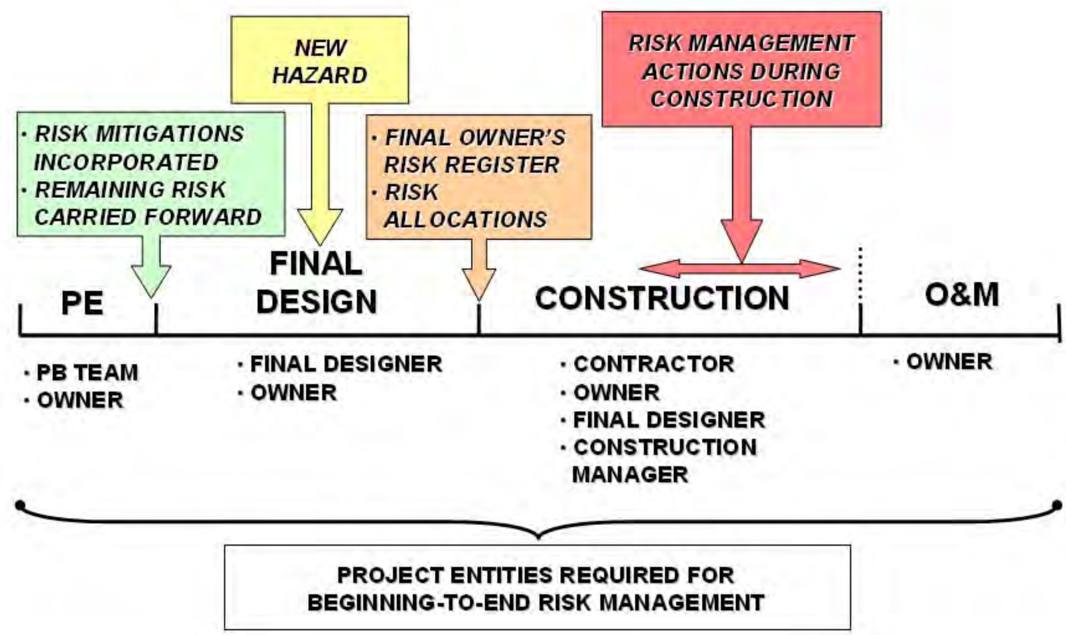


Figure 14-9 Risk Management throughout the Project Cycle

On complex projects design support technologies such as virtual design and construction (VDC) combined with risk management and risk analysis software provides added value in managing risk in the design phase and during construction. Using virtual design and construction and risk analysis models project managers are be able to visualize the impacts of unmitigated risk on the project, perform interference checking and clash detection to mitigate risk and control schedule overruns. Project managers can see or experience the project in a highly visual, consistent and interactive manner, and individual teams can drill deeper into the modeling database to evaluate specific project elements, options, layers, disciplines and construction phases of any contract package or combination of packages, that will support critical decision making and mitigate risk. By combining the attributes of VDC and risk analysis projects can avoid costly design and construction errors before they happen and improve communication and coordination during construction. A collaborative risk analysis and VDC approach to risk management takes the guess work out of the project.

Once the underground facilities are in place, some might suggest that most of the risks have been overcome, and the facility will operate for its scheduled life as planned. This is only true only if certain operational risks are mitigated. In fact, the long-term consideration of the operational risk for a tunnel sets a number of design criteria for the works.

CHAPTER 15
GEOTECHNICAL AND STRUCTURAL INSTRUMENTATION

15.1 INTRODUCTION

In the context of this manual, the primary purpose of geotechnical and structural instrumentation is to monitor the performance of the underground construction process in order to avoid or mitigate problems. If such monitoring also serves a scientific function, or leads to advancement in design procedures, that is a bonus rather than a primary reason for its implementation. A few decades ago monitoring was not a particularly easy task because the tools were few and some not so well developed. Monitoring was generally performed manually, and the refining of data to a state of usability from the raw readings often required long hours of “number crunching” with relatively crude calculators and more long hours of plotting charts and graphs by hand.

The world of the early 21st century is very different for those who pursue the art of determining what ongoing construction is doing to its surroundings, or even to itself. Advanced and refined types of instrumentation abound, and electronics coupled with computers has made remote monitoring, even from half a world away, practically an everyday affair. It is common for even medium sized projects to run a computerized database that reduces raw readings to usable data and can report on any combination of instruments and data plots within minutes. It can also inform interested parties any time of the day or night if movements or stresses have reached pre-set trigger levels that demand some kind of mitigative action. The possibilities have not gone unnoticed by project Owners, and comprehensive instrumentation and monitoring programs are becoming the norm rather than the exception. This is perhaps especially true in the world of tunneling where even small mis-steps can result in damage that may lead to lawsuits or the shutting down of operations.

Readers should be aware that much of the instrumentation described herein may not lend itself particularly well to rural highway tunnels, especially those located in hilly or mountainous terrain that may limit the need for instrumentation if great tunnel depth minimizes ground settlement at the surface, and if lack of surface development minimizes the number of third-party abutters who could be affected by construction. Also, even if a tunnel does require monitoring for whatever reason, great depth may minimize possibilities for damage to surface installations and push designers and constructors toward more in-tunnel installations.

The amazingly large number of instrument types available to tunnelers means that this chapter can do little more than “broad brush” the subject. The most common and/or most promising types of instrument will be covered, but readers will have to turn to the references to see what else is available. A few types will be covered to some degree in other chapters; for example, earth pressure cells that are commonly used by those who specialize in Sequential Excavation Method (SEM) tunneling (Chapter 9), but are not so much used by those who work in other types of underground construction. Although vibration monitoring will be covered herein, the monitoring of noise will not be covered because it is normally considered an environmental rather than a structural or geotechnical concern. Some instruments, such as those used to determine in-situ ground stresses prior to tunneling, will not be covered because they more rightly belong in the category of site investigation instrumentation. And finally, there will not be space to delve deeply into the theory of operation of the various instruments discussed, so readers will again have to turn to the referenced publications for more details.

The first few sections of this chapter will discuss the types of measurements typically made:

- Ground Movement away from the tunnel
- Building Movement for structures within the zone of influence
- Tunnel movement of the tunnel being constructed or adjacent tubes
- Dynamic Ground Movement from Drill & Blast
- Groundwater Movement and Pressure due to changes in the water percolation pattern

The first three items comprise quasi-static changes in position, and the last is also concerned with long-term effects. In contrast, Dynamic Ground Movement covers response due to vibration caused by the shock waves generated by explosive charges used to excavate rock.

All of the monitoring needs to be coordinated to fit with the tunnel construction schedule, and to establish the actions that must be taken in response to the instrumentation findings. These topics are discussed in the final section of this chapter.

15.2 GROUND MOVEMENTS – VERTICAL & LATERAL DEFORMATIONS

15.2.1 Purpose of Monitoring

The primary purpose for monitoring ground movements is to detect them while they are still small and to modify construction procedures before the movements grow large enough to constitute a real problem by affecting either the advancing excavation or some contiguous existing facility. For the advancing excavation, ground support has to be based on conditions encountered; monitoring either confirms the adequacy of the support or indicates whether more or different support may be required. Existing facilities may be at the ground surface – roads, railroads, buildings and the like – or they may be below ground in the form of utilities or other transportation tunnels such as subways. The first line of defense against potentially damaging movements is to detect them at depth in the ground immediately surrounding the advancing tunnel and take mitigative action before those movements can "percolate" upward toward the surface. This kind of monitoring can provide an indication of whether ground treatment such as grouting is effectively limiting movements that might otherwise result in troublesome settlements. Ground can, of course, move upward as well as downward, in the form of heave from unloading that can destabilize the invert of the tunnel under construction, and as a side effect lead to lateral, possibly damaging deformations as the ground moves toward the excavation to take up the slack. In addition to helping control the ground, the data developed can be used (and this may be said of all monitoring discussed in succeeding paragraphs) to verify design assumptions and to evaluate claims by construction contractors and third-party abutters.

15.2.2 Equipment, Applications, Limitations

Several types of instrumentation are used to monitor ground movement:

- Deep Benchmarks
- Survey Points
- Borros Points
- Probe Extensometers
- Fixed Borehole Extensometers, either measured from the surface or during advance of the tunnel
- Telltales or Roof Monitors
- Heave Gages

- Conventional Inclinometers
- In-place Inclinometers
- Convergence Gages

15.2.2.1 Deep Benchmarks

Deep Benchmarks (Figure 15-1) are steel pipes/casings drilled into stable strata – preferably sound bedrock – outside the advancing tunnel's zone of influence. They are used when existing benchmarks, such as those installed by the USGS, are not available and it is important to know actual elevation changes of other instruments meant to detect movements. If installed close to the construction, deep benchmarks need to be carried below invert. They must be absolutely stable in spite of any ground movements that are occurring because it is the surface level collars of these devices that become the unmoving points from which locations and elevations of other instruments can be determined by surveying. A major complication in the installation of benchmarks can be the difficulty of installing them in a location and/or to a depth that absolutely guarantees no movement as tunneling proceeds. In this regard the lowering of groundwater in a soft ground environment can contribute to ground settlements well outside the immediate projected footprint of the advancing tunnel, so the instrument has to be well placed to guard against this eventuality. In cases of very large projects or overlapping projects that cause the water table to be drawn down across a large area, benchmarks have been known to settle even when founded in bedrock because some rock types can be dependent to a degree on pore water pressure for their ability to carry load.

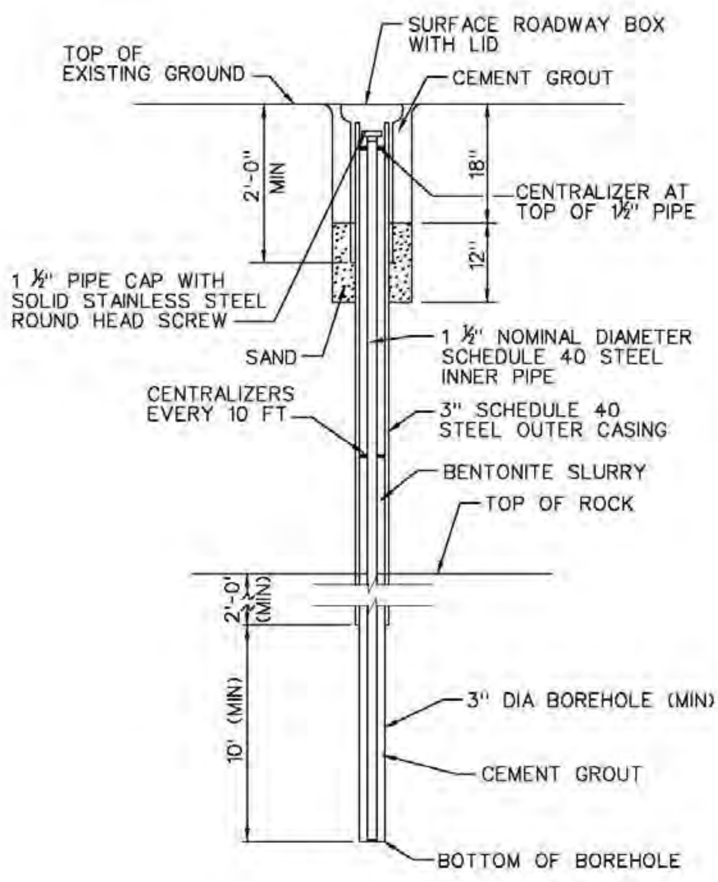


Figure 15-1 Deep Benchmark

15.2.2.2 Survey Points

Survey Points are used to detect ground movements at the surface or a few feet below the surface. They may be as simple as wooden stakes driven into the ground and their elevations surveyed through backsighting to a deep benchmark (Figure 15-2). Penetration needs to be at least a foot or so to guard against dislodgment, and the tops should not extend high enough to interfere with mowing machines if they are in a grassy area that requires routine maintenance. A survey point may also be somewhat more sophisticated and take the form of a steel rod with a rounded reference head driven several feet into the ground for better avoidance of possible dislodgment and surface effects such as frost heave (See Figure 15-3). This type of point needs to be protected at the surface by a small utility type roadbox with a secured cover so there is no disturbance to the rounded head. A rounded head is considered best because a surveyor can then always find the high point that has been surveyed in the past for good continuity in the readings. Because there is no hard connection between the rod and the roadbox – the one sort of "floats" inside the other – the survey point is also protected from being pushed down in case of the passage of a heavy vehicle. The major concerns with any type of survey point is the need to keep it out of the way of other users of the area and also protected against damage that may require replacement and lead to loss of continuity between the latest reading and the string of readings taken in the past.

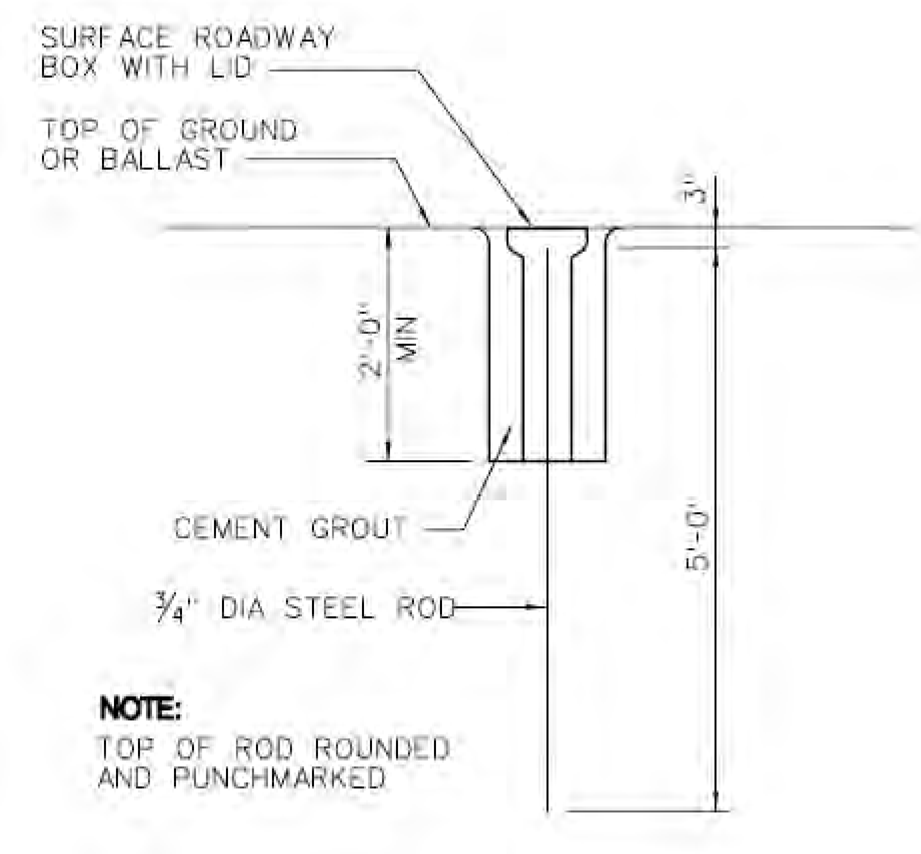


Figure 15-2 Survey Point

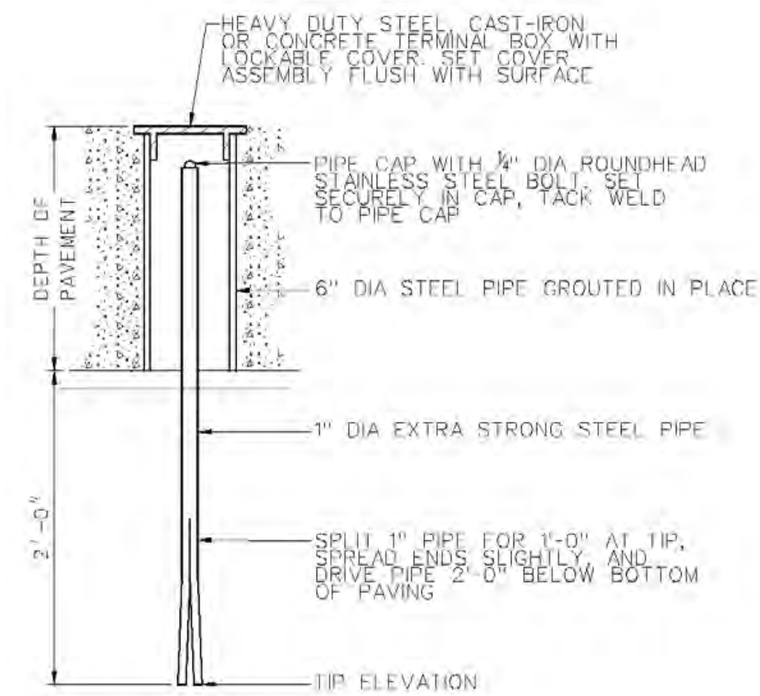


Figure 15-3 Survey Point in Rigid Pavement Surface

15.2.2.3 Borros Points

A Borros Point is basically an anchor at the lower end of a driven pipe (See Figure 15-4). The anchor consists of three steel prongs housed within a short length of steel pipe with points emerging from slots in a conical drive point. Installation is achieved by advancing a borehole in soft ground to a few feet above the planned anchor depth and the anchor inserted by attaching extension lengths of riser pipe and outer pipe. When the point reaches the bottom of the hole, it is driven deeper by driving on the top of the outer pipe. The prongs are then ejected by driving on the riser while the prongs are released and the outer pipe bumped back a short distance to achieve a positive anchorage. Such installations are useful for determining the amount of settlement at one precise depth with more certainty than the simple driven steel rod described above, and they are relatively simple and economical. The amount of anchor movement is determined by surveying or otherwise measuring the movement of the inner riser pipe at the ground surface. One disadvantage with such movement detection (and this can be said of most instruments whose data depends on movements measured in a surface mounted reference head) is that, if settlement is great enough to have affected the surface at reading time, then the whole instrument may be moving downward by a certain amount while the anchor is moving downward by a greater amount. Absolute anchor movement may then be difficult to judge unless ground elevation surveys are undertaken at that time and the changes added to the apparent anchor movement.

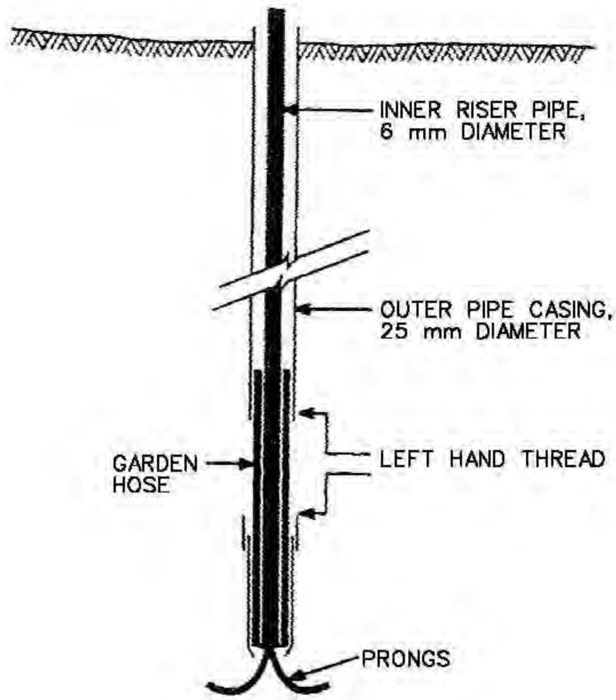


Figure 15-4 Schematic of Borros Point (After Dunnicliff, 1988, 1993)

15.2.2.4 Probe Extensometers

Probe Extensometers are used to measure the change in distance between two or more points within a drilled hole in soft ground, through use of a portable probe containing an electrical transducer. As shown in Figure 15-5, the probe, which contains a reed switch, is inserted into a casing in the drill hole in which the reference points, each of which contains an array of bar magnets, have been fixed in a way to surround the casing on the outside. In the most common type of installation, the reference points are held in place by spring loaded anchors – leaf springs – that ”bite” into the ground. The points are free to move with the ground because the outer support casing will have been removed and replaced by grout. The probe detects the depth of the reference points for an indication of whether the soil at those depths is settling due to disturbance from construction. A probe extensometer can thus measure the settlements at a much larger number of depths than can a Borros Point. Probe extensometers are generally drilled to a depth below any potential zone of influence near a cut-and-cover or mined tunnel. The bottom reference point then becomes the unmoving reference from which the movements of the shallower points are judged. In a typical situation near a mined tunnel, it is likely that the lowest moving point will exhibit the most settlement, and that settlements will prove to be less as the probe moves up the casing to where the settlement trough is widening. One problem with probe extensometers is that collection of data can be operator sensitive as the instrument reader strains to detect the exact location of the probe at each reference point depth by listening for the electronic “beep” to ensure readings at precisely the same spot time after time. Another concern may be the time required for monitoring, especially if a large number of reference points have been installed, because the probe does have to be lowered to the bottom of the casing and then readings collected as it is slowly winched back to the surface.

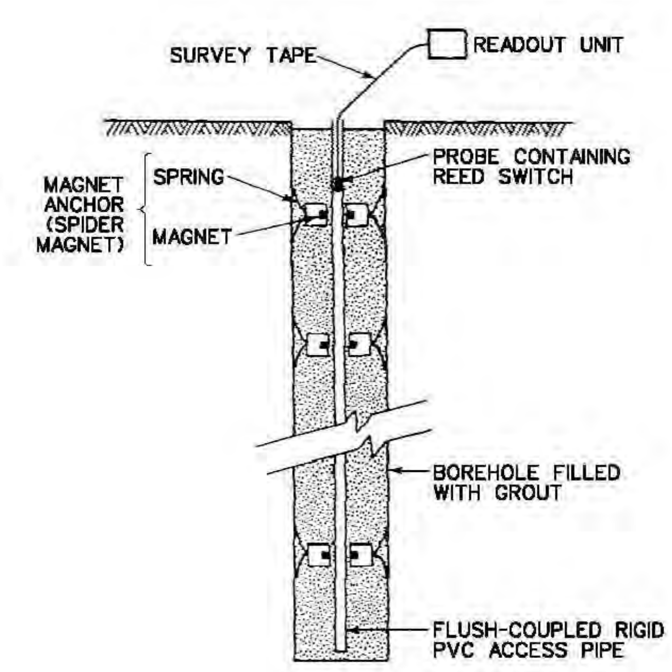


Figure 15-5 Schematic of Probe Extensometer with Magnet/Reed Switch Tansducer, Installed in a Borehole (After Dunnicliff, 1988, 1993)

15.2.2.5 Fixed Borehole Extensometers Installed from Ground Surface

Fixed Borehole Extensometers installed from ground surface may be used in soft ground or rock and may be Single Position (SPBX) for settlement measurements at one specific elevation or Multiple Position (MPBX) for measurements at several elevations. Figure 15-6 illustrates a schematic of an MPBX. The anchors of a borehole extensometer are grouted into the ground, commonly at various distances above the crown of an advancing tunnel, and connected to surface mounted reference heads by small diameter rods of steel or fiberglass. By detecting movement of the tops of the rods at the surface, one can tell how much each anchor – and hence its increment of soil or rock – is moving in response to excavation and so take steps to mitigate developing problems. Manual readings can be taken in a matter of minutes, assuming there is no problem with access to the instrument collar. However, automatic readings with an electrical transducer and datalogger – which can be salvaged/moved for use on other instruments – are relatively inexpensive and can provide real time data that feeds directly and quickly into a computer for fast analysis and databasing. Although extensometers oriented vertically over mined tunnel crowns are the most common installations, two others may prove useful in particular situations: (a) instruments angled in toward tunnel crowns or haunches from sidewalks where vertical installations are precluded by heavily travelled roads; and (b) instruments installed along the sidewalls of mined tunnels or cut-and-cover excavations where a knowledge of the vertical component of overall ground movement may be advantageous. A common problem with manually read instruments is the one of operator sensitivity, and if more than one reader is employed, they need to practice together to make certain they can monitor with good consistency. Remote monitoring leads to the concern that data collectors and analyzers may, without themselves personally having an eye on the construction operation, be unaware of the type and scheduling of activities that are affecting the data. Hence it may be necessary to make arrangements for construction progress reports to be delivered on a tighter schedule than otherwise might be necessary.

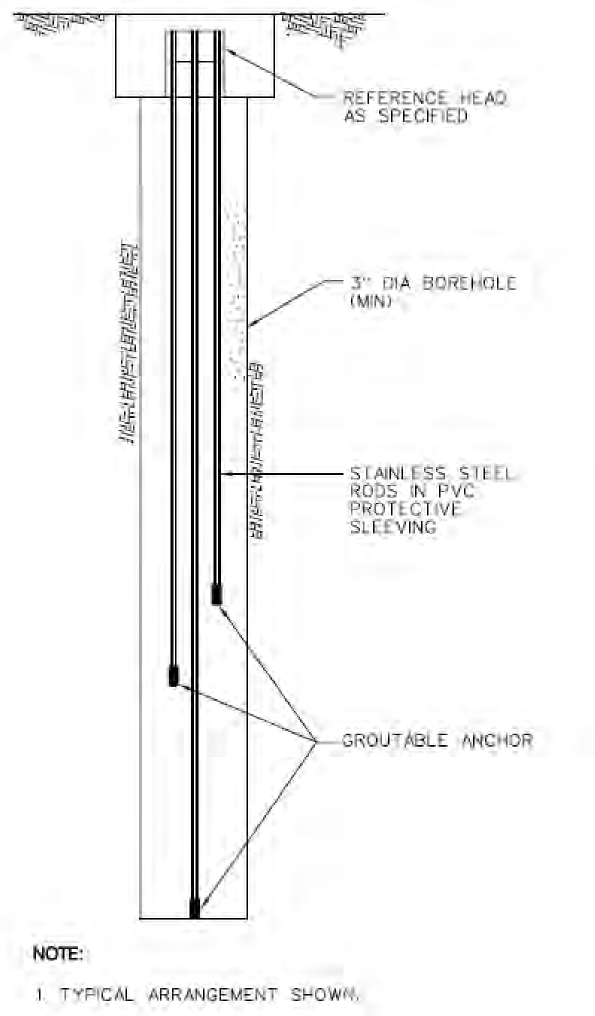


Figure 15-6 Multiple Position Borehole Extensometer Installed from Ground Surface

15.2.2.6 Fixed Borehole Extensometers Installed from Advancing Excavations

Fixed Borehole Extensometers installed from advancing excavations are a fairly obvious need if sidewall movements are required for a cut-and-cover excavation. Such horizontal installations are common and the drilling/installing operation has to mesh with the construction so that the larger operation is not overly impacted by what may appear to be a peripheral activity. (Note: "Horizontal" installations are seldom truly horizontal because angling downward by 10 or 15 degrees makes it much easier to manage the grouting of the anchors.) The installation of extensometers oriented from the vertical to the horizontal – including all angles in between – from inside advancing mined rock tunnels may be mandated by the lack of access from the ground surface (Figure 15-7). If possible, they are normally installed just behind a tunnel working face or the tail shield of a TBM. In this position they can provide data on incipient fallouts or more subtle rock movements toward the opening. If installed where a small tunnel is to be enlarged to greater size at a later time, the instrument heads can be recessed beyond the initial excavation outline and saved for use in monitoring the larger excavation. In this way they provide an almost complete history of rock movements from the earliest to the latest point in time. Another way to use these instruments is to install them from a first driven tunnel toward the location of a following twin tunnel. Readings then indicate whether the pillar between the two tunnels is loosening so that steps can be taken to mitigate the problem.

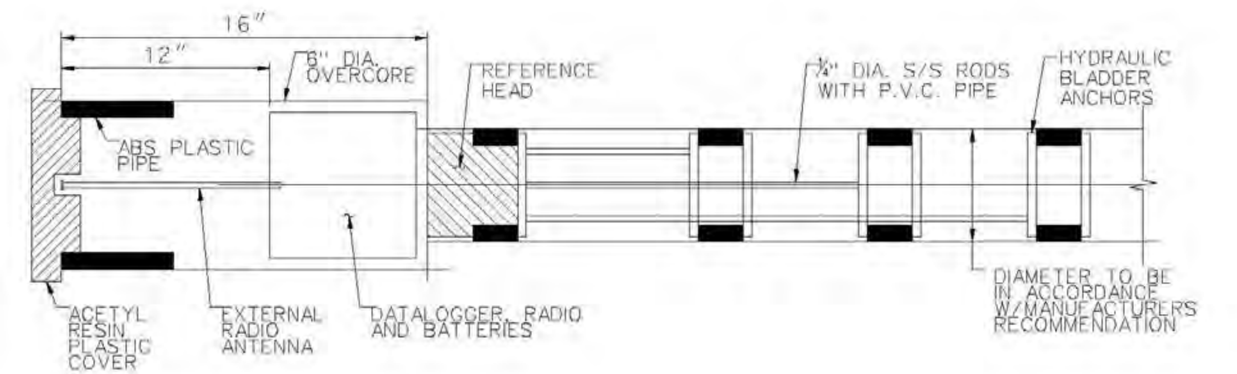


Figure 15-7 Horizontal Borehole Extensometer Installed from Advancing Excavation

Complications for these in-tunnel instruments are more numerous than for those installed from somewhere outside the excavation. As noted, the installation has to be meshed with the construction operation, a particularly tricky proposition in the confines of a small mined tunnel, where constructor complaints of interference are extremely common. Even the collection of data, if it is performed manually, may be obtrusive, especially if cessation of tunneling, use of ladders, or help from constructor personnel are involved. Remote monitoring is also possible, but then there is electrical wiring to be run and the need to find a place for the datalogger(s) to be out of the way. By whatever method the in-tunnel instruments are monitored, the reference heads need to be protected, often by countersinking them in the tunnel wall and perhaps through installation of protective covers. This is especially true where there is going to be more blasting in the vicinity, but also true even where blasting is not involved. Miners tend to have little reverence for objects whose importance is not obvious to them, so vandalism and theft of instrument accoutrements has to be guarded against. Finally, there is the fact that an in-tunnel instrument is almost always installed after the tunneled ground has started to relax, so the initial readings are seldom true zero points from which to compute follow-on movements. The instrumentation specialist's only recourse is to continually press the constructor for access to install instruments at the earliest possible opportunity.

15.2.2.7 Telltales or Roof Monitors

Telltales or Roof Monitors (Figure 15-8) are other devices that can be installed from inside an advancing rock tunnel. They are designed to be installed with anchors in stable rock beyond the tips of rock bolts in tunnel roofs to provide fast feedback on stability. The immediate safety of the miners/tunnelers is the primary reason for the instrument's use. The devices were pioneered in French coal mines in the 1970s and further refined by the British and others in succeeding years. The first ones were steel rods with a single anchor and visual movement indicators in the tunnel roof that could be seen by miners as they worked. Simple and installable by rock bolting crews, they proved vulnerable to shearing due to movement of rock blocks and were eventually replaced by more flexible steel wires that are less prone to failure. Modern versions have as many as three anchors and can be wired for remote reading by a trained person watching the data on a laptop computer. Roof monitors are widely used around much of the world and are gaining acceptance in the U.S., where they deserve to join the ranks of commonly used instruments. They are now used in civil as well as mine construction and also in rock other than flat lying sedimentaries commonly associated with coal seams. As of this writing, the primary factor in considering use of roof monitors in the U.S. may be the need to educate tunnel designers and constructors in their efficacy and ease of use.

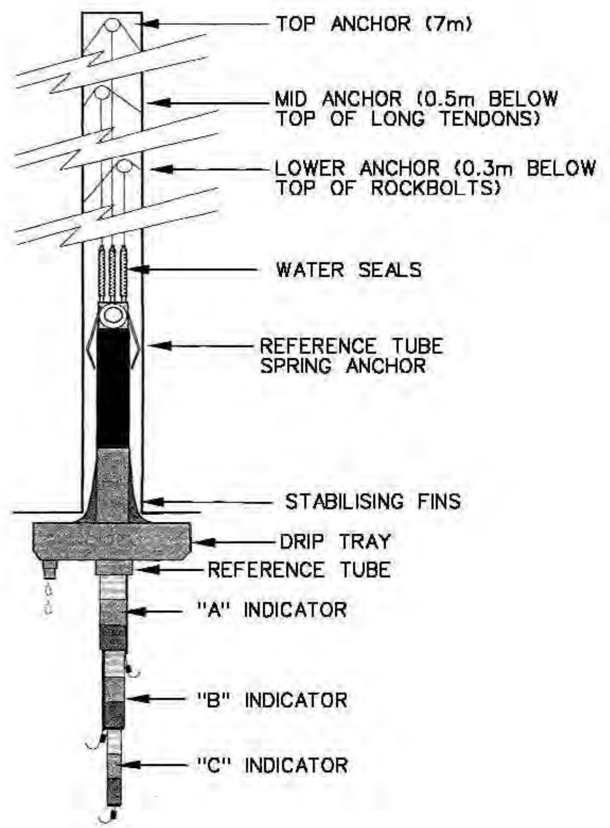


Figure 15-8 Triple Height Telltale or Roof Monitor

15.2.2.8 Heave Gages

Heave Gages are most commonly used when excavating for open cut or cut-and-cover in soft clay where there is potential for the bottom to fail by heaving as overburden load is removed. There are several instruments with which heave can be detected and measured, but almost all either suffer from lack of accuracy or are prone to damage or malfunction. Interestingly, the magnet/reed switch gage packaged as for a probe extensometer is probably the best alternative (Figure 15-9). In this type of installation the user measures increasing rather than decreasing distances between spider magnets and a fixed bottom anchor. With care taken to make certain the bottom anchor is well below any expected zone of movement, the installation is made inside the cofferdam prior to start of excavation. After initial readings are taken the access pipe is sealed 5 to 10 feet below the ground surface through use of an expanding plug set with an insertion tool, and the pipe is cut with an internal cutting tool just above the plug. A good fix is made on the plan location of the instrument and, just before the excavation reaches the plug, the pipe is located, a reading made, and the pipe again sealed and cut. The procedure is repeated until excavation is complete. The concern with such installations – a concern not overcome with alternative installation types – is that any large excavation is made by means of heavy equipment, and operators are not prone to watching and caring for things as small as a heave gage pipe. It is common for the gages to be damaged beyond use, and their protection can be assured only through some forceful construction management and sometimes the levying of penalties for instruments damaged as a result of contractor carelessness.

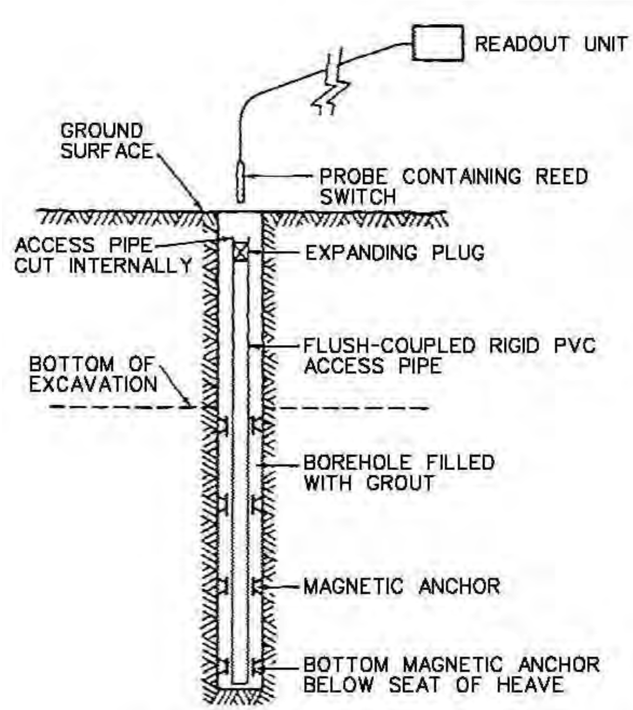


Figure 15-9 Heave Cage

15.2.2.9 Conventional Inclinometers

As shown in Figure 15-10, conventional Inclinometers are aluminum or plastic casings drilled vertically to below the level of construction into a stable stratum and used to determine whether the surrounding ground, either rock or unconsolidated material, is moving laterally toward the excavation. Each casing has tracking grooves to guide the sensing probe for orientation both parallel to and at right angles to the axis of the excavation. The probe, which contains tilt sensors, is lowered on a graduated cable to the bottom of the hole and winched upward, with stops at 2-foot intervals for collection of inclination data by means of a readout unit at the ground surface. An iterative process of tilt calculations from the unmoving bottom of the casing permits plotting of a profile that fixes each measured increment of casing in space in relation to the excavation. An initial set of inclination readings is taken before excavation begins and each set of readings thereafter during construction provides data on how the ground is moving when the user plots the newer movement curves against the initial pre-construction curves. The inclinometers are normally situated a few feet from the excavation periphery of open cut or cut-and-cover excavations, but may also be installed just outside a mined tunnel where lateral movement data may be combined with vertical movement data from the extensometers discussed above. The term “conventional inclinometer” is used herein to distinguish the manually read instrument from the “in-place” instruments described below. The major concern with a conventional inclinometer is the time consumed in the monitoring process. Readings are performed twice in each monitoring visit, once with the probe inserted in the “A” direction tracking grooves, then again with the probe in the “B” direction. A “check sum” procedure is carried out by examining the sum of the two readings at the same depth, 180 degrees apart, in order to remove any long term drift of the transducers from the calculations. It commonly requires 45 or so minutes for a reader to collect data from a 100-foot deep instrument, and that is assuming no indication of excessive movements, which, if discovered, may require another set of readings for confirmation that the movements are real and not due to a reading error or instrument malfunction.

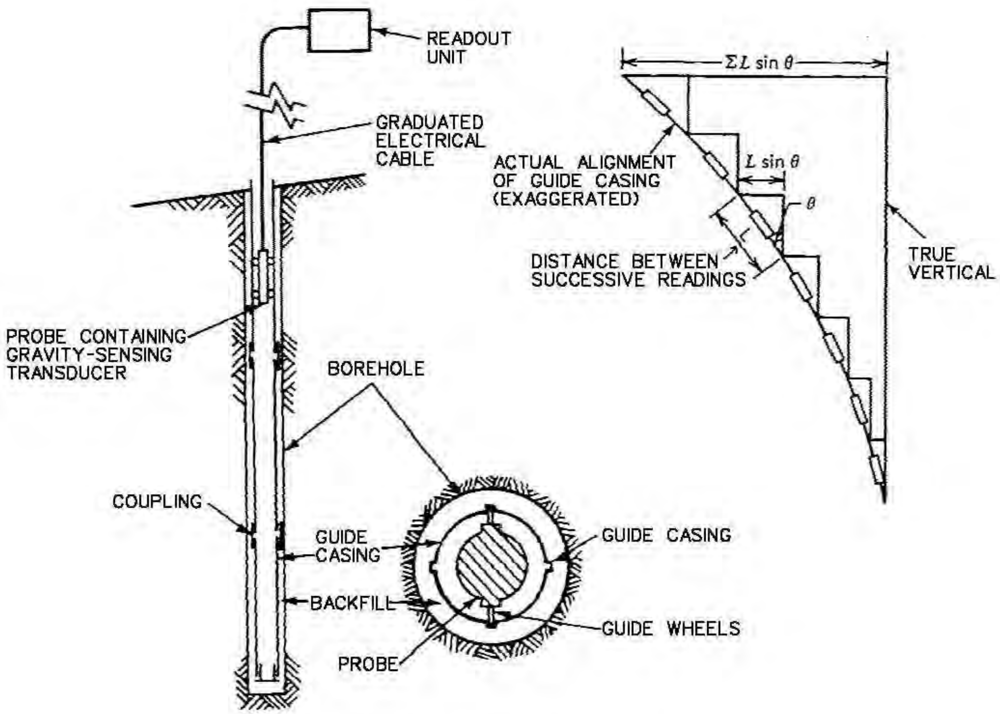


Figure 15-10 Principal of Conventional Inclinometer Operation (After Dunnicliff, 1988, 1993)

15.2.2.10 In-Place Inclinometers

In-Place Inclinometers are typically used for monitoring subsurface deformations around excavations when rapid monitoring is required or when instrumented locations are difficult to access for continued manual readings. The sensors are computer driven, gravity-sensing transducers joined in a string by articulated rods, and they can be installed equidistantly in the casing or concentrated in zones of expected movement (Figure 15-11). With the in-place instrument, as many as ten or twelve sensors are mounted in the casing and left semi-permanently in place. A larger number of sensors would be difficult to install in a standard size drill hole because each sensor has its own set of signal wires that take up space, and a very large number of sensors could result in the need for an uneconomically large diameter drill hole. Signals are fed to a datalogger at the surface and can be collected as often as required, or even fed by telephone line to the database computer for something close to real time monitoring. Compared with conventional instruments, the in-place inclinometer hardware is expensive and complex. This can sometimes be compensated to a degree by removing sensors from a bypassed instrument and installing them in a new location as the excavation progresses. A not-so-easily-overcome disadvantage of the in-place instrument lies in the fact that, if there is any long term drift in any of the sensors, it cannot be overcome through the check sums procedure described above. It is also true that the somewhat limited number of sensors in a standard in-place installation leads to a less smooth plot of movements compared with what can be achieved with the conventional inclinometer.

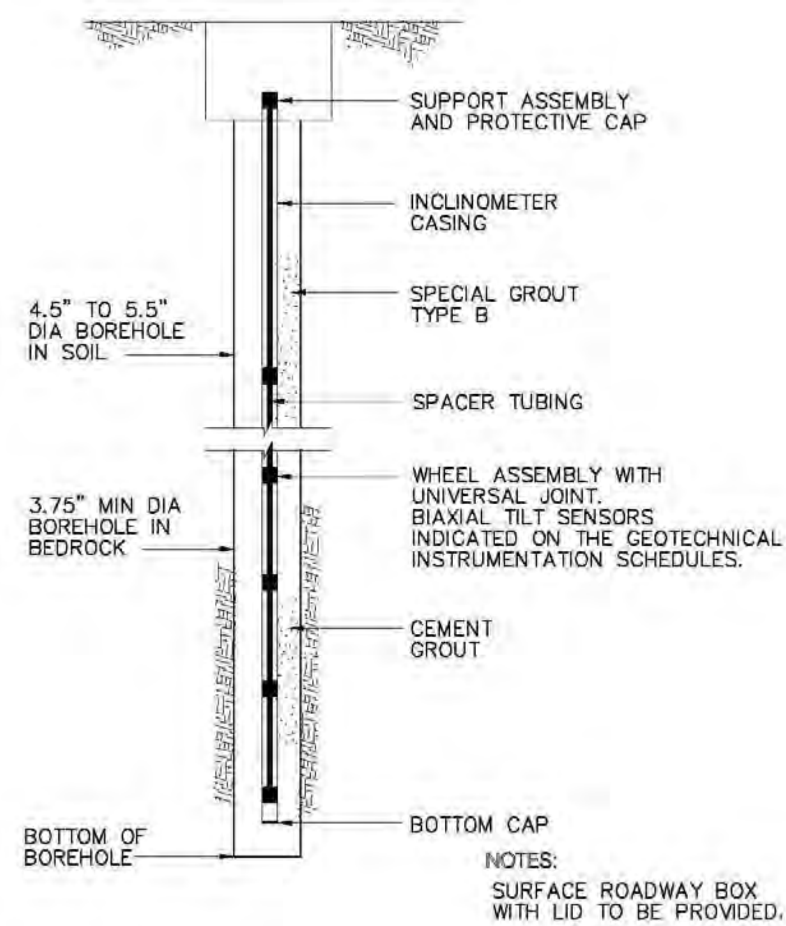


Figure 15-11 In-Place Inclinometer

15.2.2.11 Convergence Gages

Convergence Gages may be used for monitoring closure of the ground across either open excavations or mined tunnels. In the first instance they perform a function similar to an inclinometer, although with many fewer data points to give a full picture of movements. In the second function, they detect the load redistribution during and after excavation and the extent to which resulting structure/ground interaction affects the tunnel shape and the lining. Until now the typical gage has been a Tape Extensometer, which includes a steel tape with holes punched at 50 mm intervals (see Figure 15-12). Anchors that define monitoring points consist of eyelets on the ends of grouted rebar sections that extend into the ground for a foot or so (Figure 15-13). The tension in the tape is controlled by a compression spring, and standardization of tension is achieved by rotating the collar until scribed lines are in alignment. After attachment of the extensometer to the anchors and standardization of the tension, readings of distances are made by adding the dial indicator reading to the tape reading. In a typical mined tunnel the pattern of anchors includes one in each sidewall at springline level and one as close as possible to the center of tunnel crown. Three readings are taken in a tent shaped pattern and the results indicate whether the tunnel support is behaving in a predictable way. For very large tunnels, the patterns may be more like trapezoids or overlapping triangles, which requires the installation of additional anchors. Such readings are only relative readings, and if absolute elevation changes are needed, this is usually accomplished by surveying the anchor that is in the crown. (Installation directly in the high point of the crown is seldom possible because of the presence of the ventilation and other lines.)

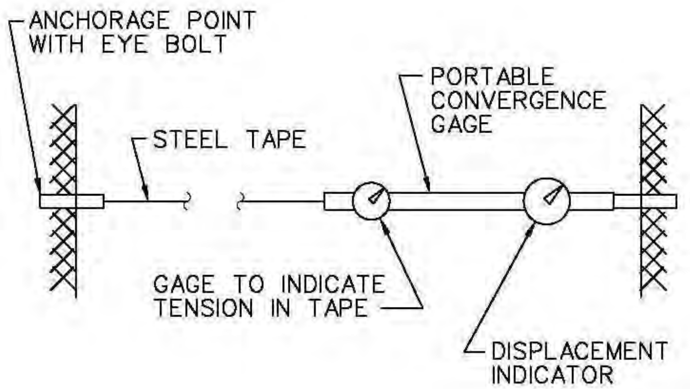


Figure 15-12 Tape Extensometer Typical Detail

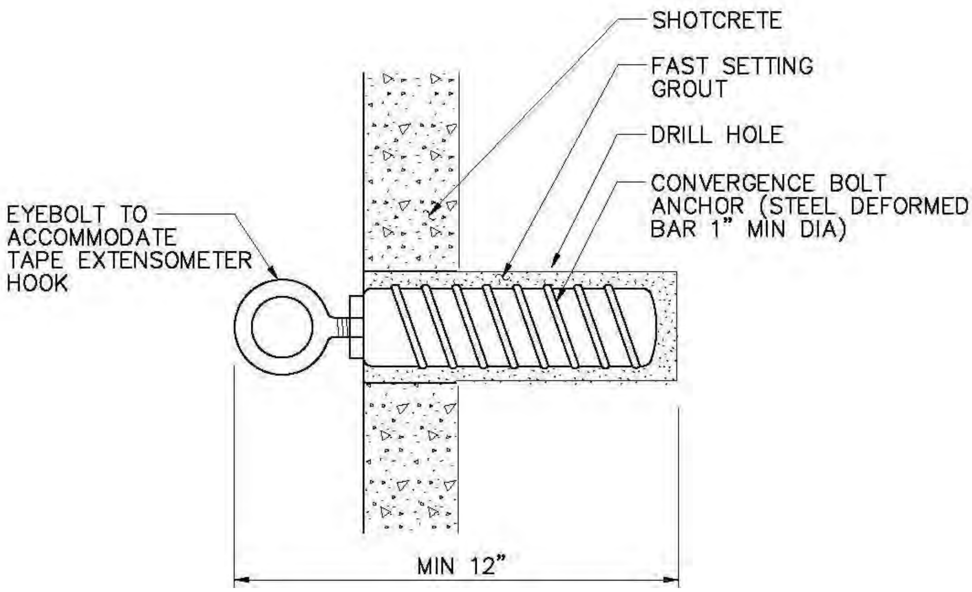


Figure 15-13 Typical Convergence Bolt Installation Arrangement

Whether the tunnel is conventionally mined or excavated by TBM, it is important to install anchors and begin readings at the earliest practicable time before the ground has begun to "work." Unfortunately, this cannot always be accomplished, especially in a TBM tunnel because, even if the anchors can be installed in a timely manner, there are scores or even hundreds of feet of trailing gear that make the stretching of a tape extensometer essentially impossible. This means that measurements may not begin until the machine is a long way past the monitoring point and knowledge of total from-the-beginning movements cannot be obtained. For this reason it seems likely that an alternative to the tape extensometer is going to be the best choice for future monitoring of tunnel convergence, and it will be in the form of a distometer. The device is small, hand held, and can be used to very accurately determine distances to a target by emitting a laser or infrared beam that is reflected from the target and detected by the same device. By installing brackets or bolts that also include targets at the places where tape extensometer eyelets would normally be placed, monitoring personnel can detect the changing shape of a tunnel without having to stretch a physical connection between points. There remains the problem that a physical object – such as TBM trailing gear – between targets will interfere with the distometer lines of sight and still not permit measurements in the standard tent shape. By judicious placement of additional brackets and targets at monitoring sections, it should be possible to gather data by working around the trailing gear in a TBM tunnel with patterns of measurements more like the afore mentioned trapezoids or overlapping triangles.

15.3 MONITORING OF EXISTING STRUCTURES

15.3.1 Purpose of Monitoring

If the different parts of a structure should move uniformly by even large amounts, damage could be minimal, maybe non-existent, except perhaps for penetrating utilities such as water pipes that might not be able to accommodate themselves to such movements. However, most structures affected by construction react by exhibiting more movement of the parts closest to the excavation than of the parts that are further away. This differential movement is the principal cause of construction related damages because the affected structure may be subjected to forces it was not designed for. A building, for example, whose footings are settling on one side while the other side settles less or not at all will suffer tilting of some walls, and the racking that ensues may cause cracking or spalling of some architectural features, freezing of doors and windows, or, in the worst case, failure of one or more of the structural members. A bridge whose footings are subjected to differential movements may undergo extensions that literally tear it apart. In general, the detection of settlements is the first line of defense in the protection of existing facilities, whether they be surface (roadways, buildings, bridges) or subsurface (utilities, transit tunnels, other highway tunnels). The detection of tilting can also be useful and has become more common as the development of monitoring devices has proceeded in the direction of increased automation. The simplest kind of monitoring involves the detection and the tracking of joint separations and crack propagation in structural concrete or architectural finishes. The ideal is to detect and mitigate some or all of these movements before they have become severe enough to cause serious damage or perhaps constitute a hazard.

15.3.2 Equipment, Applications, Limitations

As with ground movement instrumentation, there are a number of choices of instrumentation:

- Deformation Monitoring Points
- Structural Monitoring Points
- Robotic Total Stations
- Tiltmeters
- Utility Monitoring Points
- Horizontal Inclinometers
- Liquid Level Gages
- Tilt Sensors on Beams
- Crack Gages

15.3.2.1 Deformation Monitoring Points

Deformation monitoring points on roads, streets or sidewalks can be as simple as paint marks that get surveyed on a routine basis. However, paint has the disadvantage that it can be visually obtrusive, may wear off with time, and may not display a single spot that surveyors can return to time after time for good data continuity. A better alternative is a small bolt-like devise set in an expansion sleeve that can be installed in a small hole drilled in concrete or asphalt as shown in Figure 5-14. The point should have a *slightly* protruding rounded head with a consistent high point that is always findable by a surveyor as he or she searches for the same unchanging spot on which to set the stadia rod. It is important that the point not protrude too much because it might then become a tripping hazard or be vulnerable to damage from equipment such as snow plows. Although they are inexpensive to purchase and install, the ultimate cost of deformation monitoring points can grow to become relatively high if data collection becomes intensive because it depends upon the mobilization of survey crews. Also, such monitoring is not always foolproof because surveyors are not necessarily attuned to the need for that high degree of accuracy that is sought by instrumentation specialists. It is very common for data thus generated to exhibit a fair amount of "flutter," i.e., apparent up-down movements that are not real, but are only the result of inconsistencies in the survey process. Such inconsistencies may result from the too-often changing of personnel in survey crews, changes that happen commonly due to the nature of the business. Luckily, extreme accuracy is not required in much of this paved surface monitoring, so if the surveyors can reliably detect changes of one-quarter inch or so, that is often good enough.

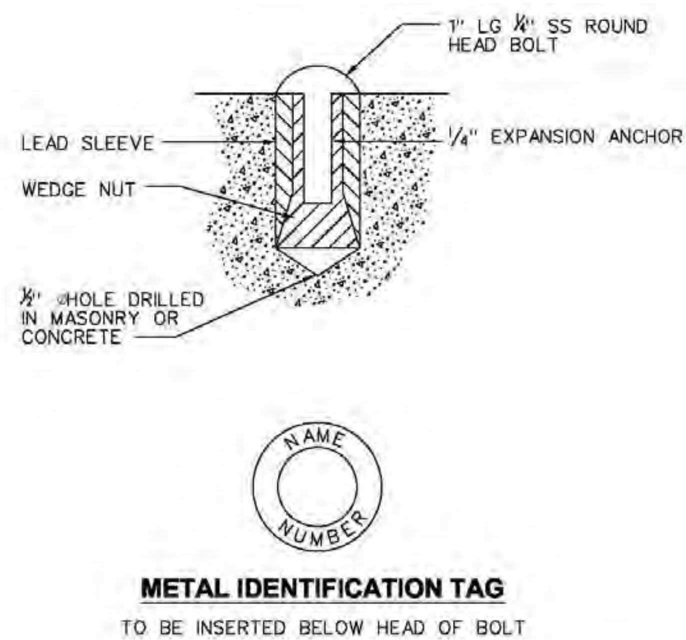


Figure 15-14 Deformation Monitoring Point in Masonry or Concrete Slab

15.3.2.2 Structural Monitoring Points

Structural Monitoring Points are survey points that are placed directly on the structures of concern, most often being installed on a vertical wall of a building or a structural element of a bridge (See Figure 15-15). Except for buildings, most structures can accommodate the monitoring point likely to do the best job and the "points" may take several forms. The simplest is a tiny scratch mark that can be easily found on each monitoring visit by a survey crew. A similar point is a stick-on decal target, which is a bit more obtrusive, but easily removable once it is no longer needed. A problem with such surface treatments is that, for buildings particularly, the monitoring point may be only on a facade that moves independently of the underlying structural elements whose movements it is important to detect. This may be overcome by the installation of a bolt-like device that penetrates to the underlying structure for a truer indication of the movements taking place. The choice of monitoring points will often depend on the wishes of owners or managers of buildings who may object to the visual obtrusiveness or potential for damage from whatever may be installed. Possible damage can extend to the post-construction period when the monitoring point may have to be removed and patched, something that is often insisted on by the party who permitted its installation. Thus, it may be necessary to repair the scars left by the removal, which may entail the use of solvents, infilling, spackling, polishing, painting or replacement for satisfactory restoration.

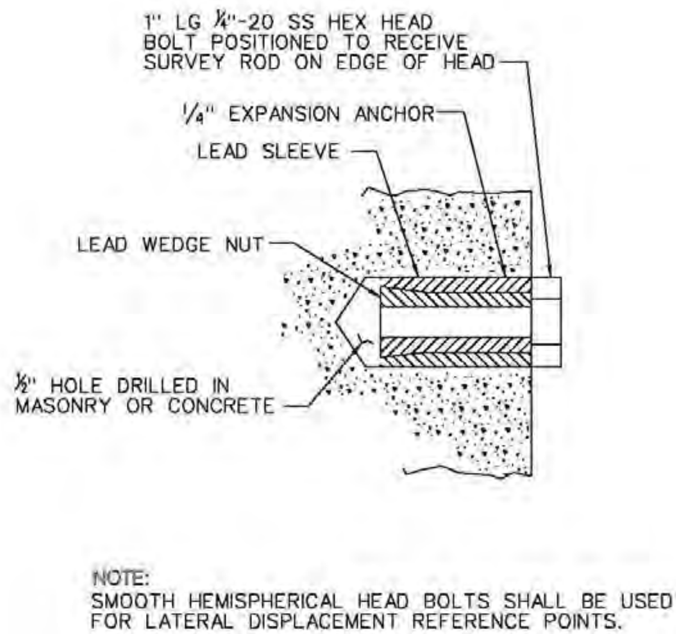


Figure 15-15 Structure Monitoring Point in Vertical Masonry or Concrete Surface

A large consideration in the use of structural monitoring points is the need to depend upon surveyors for the collection of data. Compared with roads and sidewalks, most structures have tight specifications on permissible movements (a lower mitigation-triggering level of 1/4 inch being not unusual), so surveying generally needs to be of a somewhat higher order, not necessarily as stringent as Class I, but at least done with additional care. One way to achieve this is to hold briefings in which the importance of great accuracy is instilled in the surveyors who will do the work. Another (if it is possible in the economic climate of the day) is to write and enforce the survey contract so that each group of structures is always monitored by the same crew using exactly the same equipment. In this way, the “flutter” may be reduced so as to minimize the need for instrumentation interpreters to average the peaks and valleys in determining if settlements are real or only apparent.

15.3.2.3 Robotic Total Stations

Robotic Total Stations are used for obtaining almost real time data on movements in three dimensions when it is not feasible to continually mobilize survey crews to collect data. The operation of a Total Station instrument (theodolite) is based on an electronic distance meter (EDM), which uses electromagnetic energy to determine distances and angles with a small computer built directly into the instrument. Accuracy is generally much greater than that achievable with the use of classical optical surveying. Moreover, the equipment based on EDMs is capable of detecting target movements along all three possible plotting axes, the x, the y and the z. Total stations used in geotechnical and structural monitoring are electro-optical and use either lasers or infrared light as the signal generator.

Robotic (also called automated motorized) total stations are configured to sit atop small electric motors and to rotate about their axes. As shown in Figure 15-16, they are mounted semi-permanently and, at pre-determined intervals, automatically “wake up” to aim themselves at arrays of special glass target prisms (Figure 15-17) that can provide good return signals from a variety of angles. The target prisms, which are 2 to 3 inches in diameter, are installed on structures of concern and the total station instruments installed on other structures as much as 300 feet away. It is best to have the total stations installed outside the expected zone of influence for absolute certainty of measuring target movements with accuracy. However, it is standard practice to install some of the prisms definitely outside the influence zone so that they become reference points from which the total station can determine its own position and calculate the positions of the other prisms that may be subject to movement. Clear lines of sight from total station to target prisms are a requirement so that careful planning is required for proper placement. Data is recorded by means of the total station’s own computer and may be fed to a centralized database computer by means of telephone lines or radio signal.

A major aspect of robotic total station use is the front end expense incurred. Depending upon the number purchased, the cost of top quality target prisms can range from $80 to $200 each in 2009 dollars. The total stations can cost from 30 to $40 thousand each, and they generally require the services of a specialist for the installation and maintenance. Nevertheless, for many projects where almost real time data on structural movements is necessary, this may be the only monitoring system capable of meeting all requirements.

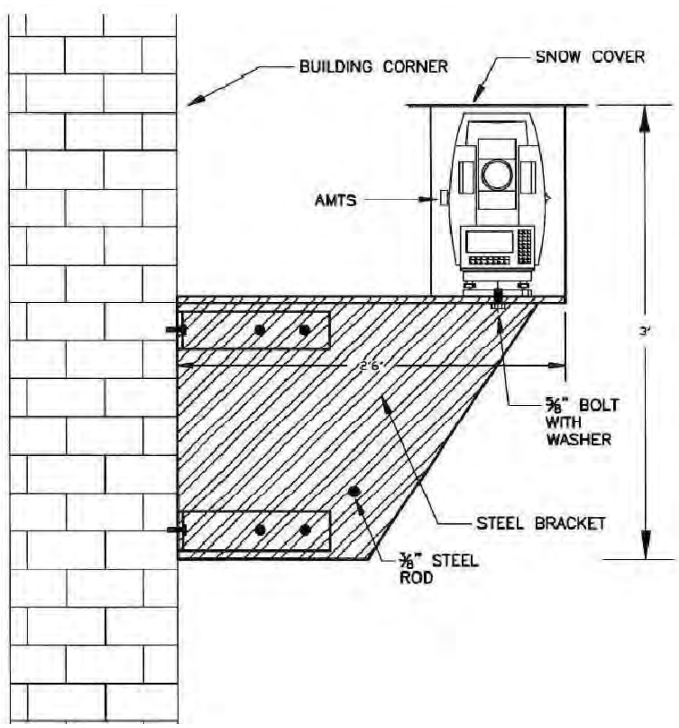


Figure 15-16 Robotic Total Station Instrument

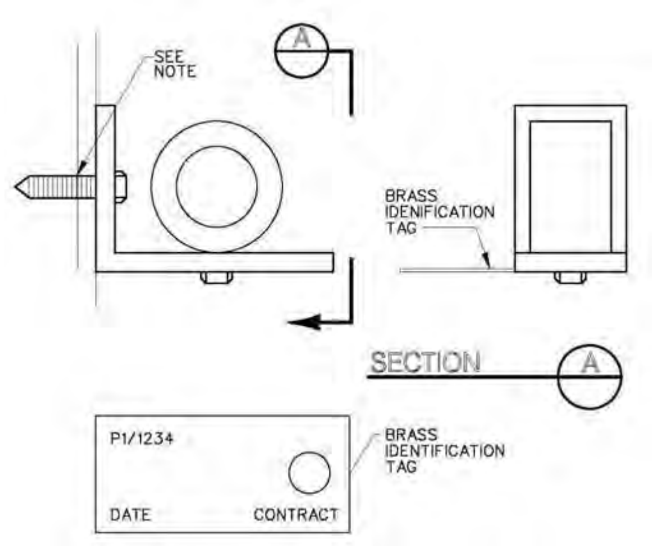


Figure 15-17 Target Prism for Robotic Total Station

15.3.2.4 Tiltmeters

Tiltmeters are used to measure the change in inclination of structural members such as floors, walls, support columns, abutments, and the like, which may tilt when the ground beneath is being lost into an advancing excavation. Manual tiltmeters generally consist of reference points on plates attached to the surface of interest and monitored by means of a portable readout unit, the functioning of which is based on an accelerometer transducer. Because such an arrangement can be operator sensitive and reading is somewhat labor intensive, especially where continued access is not easy, it is becoming more common to collect data remotely by means of electrically powered tiltmeters whose sensing elements may consist of accelerometer or electrolytic level transducers placed in housings that can be attached to the element to be monitored. If only one direction of movement is expected, the chosen instrument may be uni-axial, but if there is a possibility of combinations of movement, the bi-axial instrument would need to be used. Figure 15-18 illustrates a biaxial tiltmeter. Because tiltmeters can inform users only about rotational components of movement, data must be combined with that from other instruments to determine levels of settlement that may be affecting the structure. The most difficult tiltmeter installations are those required for structural elements somewhere inside a building that is occupied. Even the manually read instrument, with a flat 6 to 8-inch diameter plate being the part attached, is somewhat visually obtrusive and may be objected to by a building manager. Remotely read tiltmeters are even more obtrusive because they need to be wired for electric power and connected to a powered datalogger that will probably need to have telephone connections if true real time data is needed. There is some controversy within the monitoring community about the best installation height for these instruments, with some opting for lower floors and some for higher floors where absolute wall movement – though perhaps not *tilt* per se – will be greater. The argument is often laid to rest by a building manager who will permit such installations only in basement levels to better keep them out of the way.

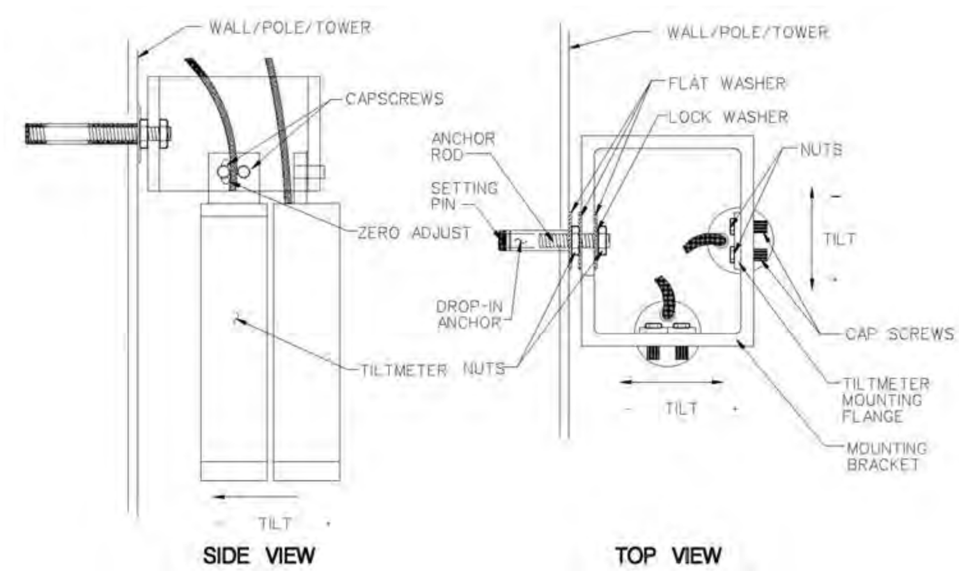


Figure 15-18 Biaxial Tiltmeter

15.3.2.5 Utility Monitoring Points

Utility Monitoring Points are very simple instruments used to determine whether an existing utility such as a water line is settling in response to an excavation proceeding nearby or underneath. The device consists of a small pipe with a rounded survey point or arrangement for use of a feeler gage at the upper end. This pipe is situated inside a larger piece of casing attached to a road box for surface protection. The lower end of the small pipe is attached to the top of the utility to be monitored and data collected by determining whether the top seems to be moving downward.

Unfortunately, such an instrument works well only if the monitored utility is exposed in a trench, and the inner pipe of the instrument attached before the utility is re-covered with backfill. When such an installation is attempted with a utility that is not exposed, one of two things may happen: (a) because the location of utilities is seldom known with absolute certainty, there is danger that the installing drillers may penetrate the utility, leading to a larger problem than the new tunnel under construction would have created; and (b) in the confines of a small drill hole it is extremely difficult to actually attach the monitoring pipe to the top of the utility, so it is possible for the utility to settle without there being an indication from the instrument of the movement's true severity.

In a case such as this, the best fallback position is to install a Borros Point (Figure 15-4) or an SPBX beside and to invert depth of the utility. If ground movement is observed at that location, it may be an indication that excavation procedures need to be modified to contain a problem. Depending upon its size and stiffness, a utility may be able to bridge over a zone of disturbance and so be in no immediate danger, but ground settlement of a certain magnitude can be an indication that the movement needs to be arrested before it does become serious.

15.3.2.6 Horizontal Inclinometers

Horizontal Inclinometers are simply inclinometers turned on their sides and the transducers in the probe (conventional instrument) or sensors (in-place instrument) mounted such that the sensitive axes are perpendicular to the length of the pipe (Figure 15-19). In this way, an inclinometer is measuring the vertical rather than the lateral movements of the instrumented structure. One use for a horizontal inclinometer is in the determination of settlement of a utility along a reach that requires continuous data not producible by the utility monitoring points or extensometers described above. Due to difficulty of continuous access for monitoring, such an inclinometer installation is more likely to entail an in-place instrument that can be remotely read, but even here access may pose at least a minor challenge. If the utility is large and the flow of contained liquids can be controlled, then inclinometer casing may be strung and attached to the roof inside the instrumented structure. If the utility is too small for entry or the liquids cannot be controlled, then it would need to be exposed in a trench for instrument attachment to the outside and then backfilled. In either case, arrangements would be made for wiring to be run to a datalogger for essentially real time monitoring. Difficulty of access for installation is an obvious drawback, but when the need for monitoring is over, it should always be possible to salvage the expensive sensors for re-use.

If entry into the utility were possible for installation, then it should also be possible for recovery efforts. If the instrument were installed and then covered over by backfill, a small manhole will have been provided for access to the reference head and the wiring, and it is from here that the sensors and their attached wires can be removed.

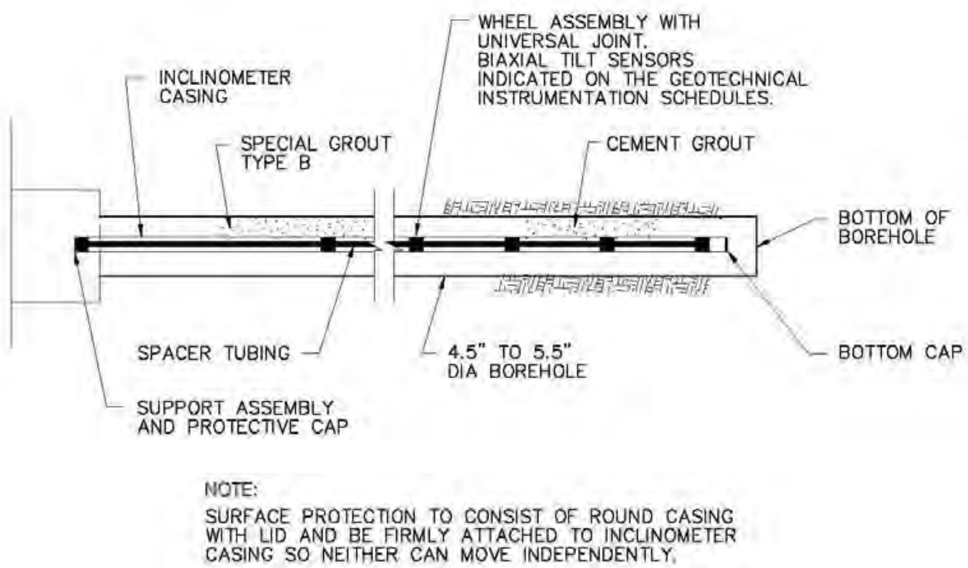


Figure 15-19 Horizontal In-Place Inclinometer

15.3.2.7 Liquid Level Gages

Liquid Level Gages are systems of sensors installed in an array that measures the height of a column of water within each gage as shown in Figure 15-20. Sensor gages are connected by small 1/4 to 1-inch diameter tubes or pipes to a reference gage outside the zone of influence. The reference gage is actually a reservoir, with its contained liquid generally kept under pressure to avoid the undesirable effects of barometric changes. The liquid completely fills all of the tubes throughout the array of components, none of the liquid is exposed to outside atmosphere, and so it is referred to as a closed pressurized system. With the liquid always at the same elevation, settlements of the instrumented locations are indicated as the heights of the columns of water within the gages change in relation to the gage housings, which are moving. Signal outputs are most commonly driven by LVDTs (see description under electrical crack gages below) or vibrating wire (see surface mounted strain gages under 15.4.2) force transducers. The closed systems are small and flexible and can be configured to fit into the convoluted layouts of many instrumented structures. Readings are collected remotely through wiring of the system to a datalogger. Such systems are commonly installed in or on a structure where continuous settlement measurement to an accuracy of several millimeters is needed and where continued access for maintenance is not a large problem.

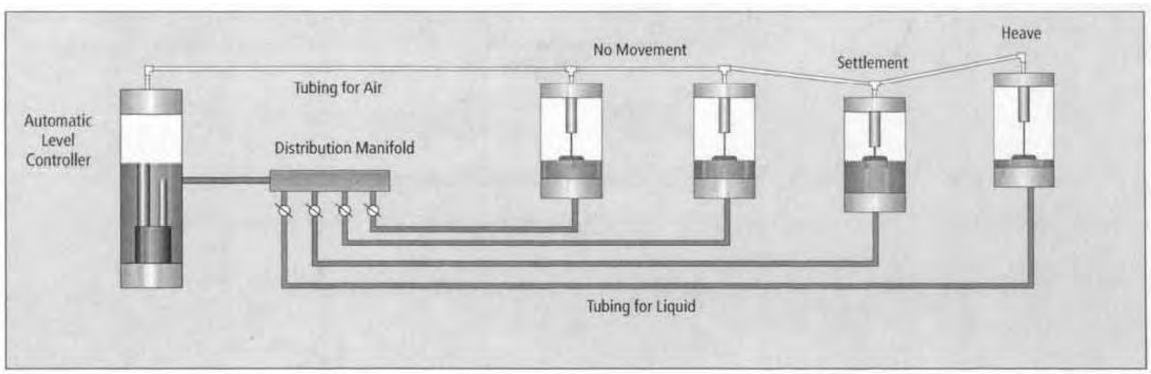


Figure 15-20 Multipoint Closed Liquid Level System

Maintenance visits are a must with these systems, and so the issue of access has to be taken seriously. During installation, which must be performed with great care, the system has to be charged with de-aired water and then purged to make certain no air bubbles have intruded to remain within it. This is one reason most installations utilize some kind of semi transparent plastic tubing; it permits visual detection of bubbles and makes purging them easier. This is critical because air bubbles will migrate to high points in the tubing or to the sensors themselves and can cause readings to be very inaccurate or can even shut down the system altogether. Then, during operation, it is very common for bubbles to appear in spite of careful installation. This may occur due to leakage from the outside, tiny amounts of air coming out of solution and accumulating, etc. Interestingly, the pressurization of the system can inhibit the emergence of bubbles, but never stop it entirely. No closed system is immune to this problem and maintenance visits may be required for purging and de-airing as often as every 6 to 8 weeks. This is why continued access can be so important to the closed pressurized system's functionality.

The maintenance problem can be largely overcome through the use of an open channel system which consists of sensors connected by pipes that are only half filled with water as shown in Figure 15-21. Open to the atmosphere, neither the liquid nor the sensors are affected by the problem of air bubbles. They can be installed to lengths of several thousand feet, operate for many months with hardly any maintenance, and still detect movements to sub-millimeter accuracy. However, such systems are large, heavy (due to the piping), sometimes difficult to install in structures with complicated layouts, and are much higher in front end costs than the smaller closed systems. At present, only a few open channel systems have been installed in the U.S. and only one or two corporate entities have expertise in their manufacture and installation. It seems likely that they will have a much larger presence in the future if downsizing of the components can lower purchase prices and make installations faster and easier.

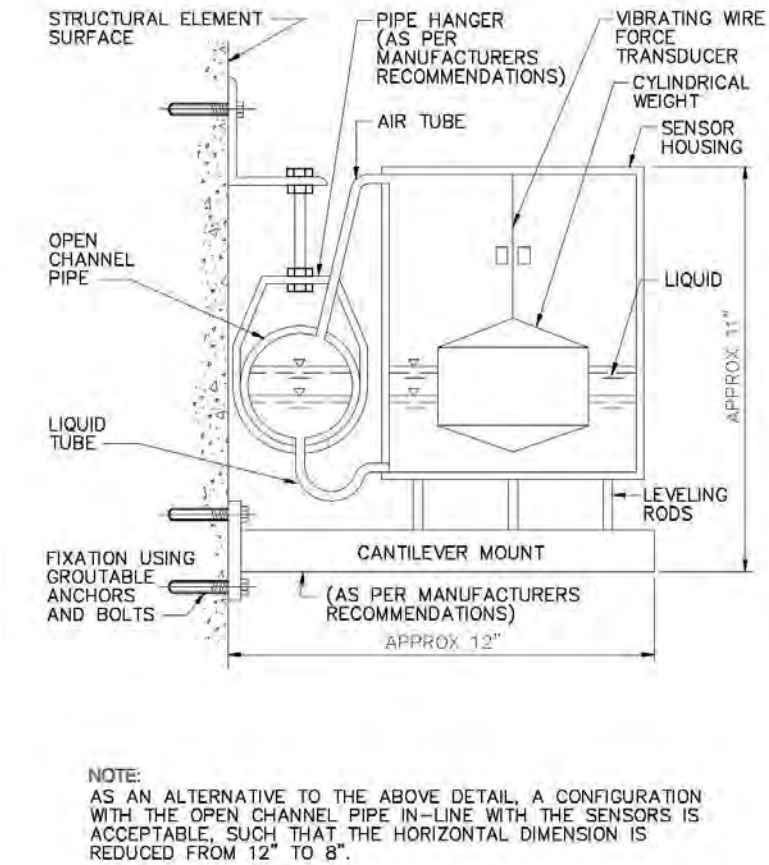


Figure 15-21 Open Channel Liquid Level System

15.3.2.8 Tilt Sensors on Beams

Tilt Sensors on Beams, when packaged to monitor elevation changes rather than tilt per se, consist of sensors attached to metallic rods or beams, with the beams linked together with pivots (Figure 15-22). By monitoring changing tilt of each sensor and knowing the length of each +/- 5-foot long beam, users can calculate elevation changes of each pivot with respect to the datum. The relative tilt of each sensor and beam is set in the field and elevation change data determined by making an initial scan of readings, called the reference set, and mathematically subtracting readings in that scan from each subsequent scan. All elevation change data is referenced to one end of the system defined as the datum. Ideally, the datum is in a stable area not likely to move, and its absolute elevation is generally determined by an initial optical survey. Integrating the data is an iterative process as settlements are computed from sensor to sensor. Readings are collected by having the system connected with a datalogger for almost real time monitoring.

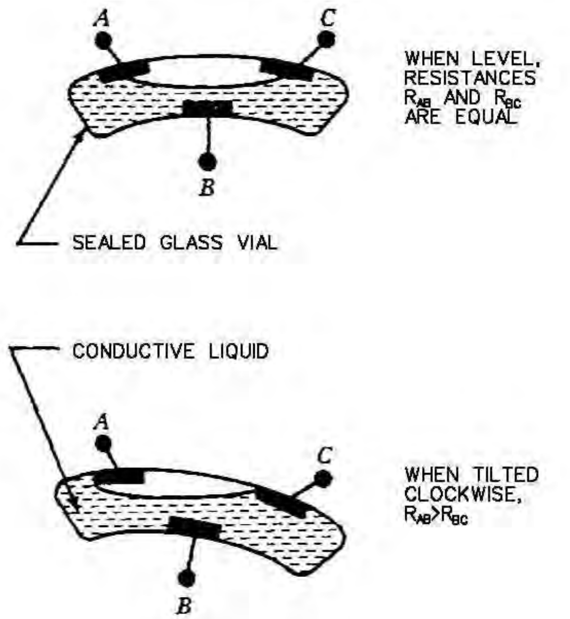


Figure 15-22 Schematic of Electrolytic Level Tilt Sensor (After Dunnicliff, 1988, 1993)

Such installations can work on bridges, the balustrades of buildings, the walls or safety walks of existing tunnels, or even railroad tracks. However, they do depend upon sensing of the mechanical movements of a string of components, and the components need to be as free from interference as possible. If installed where workers or moving equipment may be present, they have to be protected by installation of metallic housings or half rounds of heavy plastic casing. Another potential problem stems from changing temperatures, especially in the outdoors where there may be exposure to severe or very changeable weather. Although the sensors may fare as well as they would in any other type of installation, such as in a tiltmeter housing, the beams and the pivots are metal and subject to thermal effects with the potential to skew the data in unexpected ways. Users need to be aware that, if even one sensor or sensor/beam combination fails for any reason and requires replacement, the whole string of sensors and beams will need to be reinitialized.

15.3.2.9 Crack Gages

Crack Gages (also sometimes called Jointmeters) as installed on structures are typically used for monitoring cracks in concrete or plaster, or for determining whether movement across joints is exceeding a structure's design limits. The first appearance of cracks can be an indication of structural distress, and their growth, either in width or length, can be an indication that stress is increasing, as can the continued widening of an expansion joint. There are several ways of measuring these movements; only the two most common can be covered herein.

As shown in Figure 15-23, a Grid Crack Gage consists of two overlapping transparent plastic plates, one installed on each side of the discontinuity and held in place with epoxy or mounting screws. Crossed cursor lines on the upper plate overlay a graduated grid on the lower plate. Movement is determined by observing the position of the cross on the upper plate with respect to the grid. Data is kept in notebooks and has to be keypunched into a computer if needed for an electronic database. Such gages are inexpensive to purchase and install, but readings may vary with changes in monitoring personnel and this has to be guarded against. There are three circumstances in which such simple devices may prove inadequate: (a) where cracks are too narrow or are widening too slowly for the human eye to detect their growth; (b) where continued physical access is very difficult and remote monitoring is required; and (c) where something close to real time monitoring is required. Such difficulties may be overcome through the selection and installation of Electrical Crack Gages as shown in Figure 15-24.

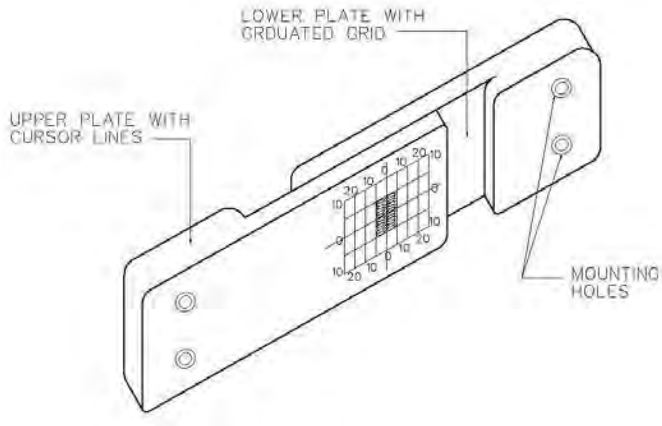


Figure 15-23 Grid Crack Gauge

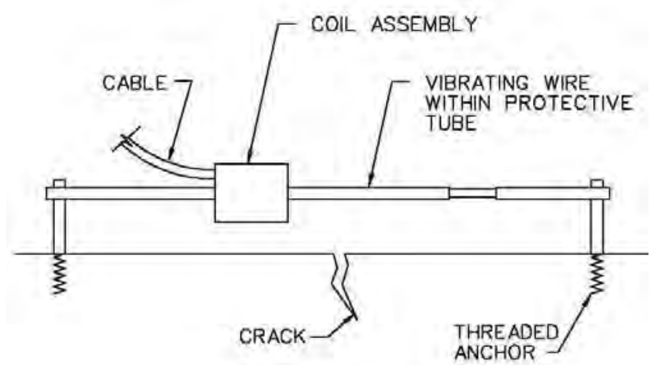


Figure 15-24 Electrical Crack Gauge

There are a number of electrical gage types, but most are based on an arrangement of pins attached on opposite sides of a joint or crack, with the pins connected by sliding extension rods whose differential movements are detected by a built-in transducer. The most common transducer is the linear variable displacement transformer (LVDT) that consists of a movable magnetic core passing through one primary and two secondary coils. Data readouts depend upon detection and measurement of differences between voltages generated in the secondary coils, magnitudes of which depend on the proximity of the moving magnetic core to the secondary coils. Users may prefer to pick up the gage signals by using a small low power radio transmitter installed at the instrument location to avoid the transmission of alternating currents through long lead wires that can introduce output-degrading cable effects.

15.4 TUNNEL DEFORMATION

15.4.1 Purpose of Monitoring

When the temporary or permanent structural support for a tunnel is being designed, calculations are performed to predict the kinds of movements and stresses the support can safely be subjected to before there is danger of failure. It is the job of instrumentation specialists to track those movements and stresses and provide guidance on whether the support or the construction process needs to be modified to ensure short term safety and long term stability of the completed tunnel. For braced excavations it is standard practice to measure the loads on some of the support members, and often to combine these with measurements of the support member deflections if the measurement of ground movements outside the support system are not sufficient to present a complete picture of support performance. It is possible to thus monitor the significant performance related behavior of soldier piles, slurry walls, struts, tiebacks and other elements of open cut or cut-and-cover excavations. In mined tunnels it is generally more common to use deflection measurements as a first line of defense against adverse developments because the eccentricities in the movements of many support members, such as steel ribs, make stress and load measurements much more complicated and prone to varying interpretation than they are for braced excavations.

15.4.2 Equipment, Applications, Limitations

Monitoring of the tunnel itself is similar to ground movement monitoring, using the following instrumentation:

- Deformation Monitoring Points
- Inclinometers in Slurry Walls
- Surface Mounted Strain Gages
- Load Cells
- Convergence Gages
- Robotic Total Stations

15.4.2.1 Deformation Monitoring Points

Deformation Monitoring Points (DMP) on support elements take several forms, but all have one thing in common: they are semi-permanent points to which a surveyor can return again and again and be certain of monitoring exactly the same point. A DMP may consist of a short bolt inside an expandable sleeve if mounted in a small drilled hole in concrete, such as a slurry wall (Figure 15-25), or may be the head of a bolt that is tack welded to a steel surface such as the top of a soldier pile. A DMP can be surveyed for both lateral and vertical movements to help determine whether the upper reaches of support may be “kicking in” or perhaps settling downward as the ground moves. If mounted in or on a vertical surface, the bolt head must have enough stick-out to permit a stadia rod to be rested on it. If mounted in or on a horizontal surface, the bolt head must be rounded, especially if it is to be used for determining vertical movements, for the same reason that a round head DMP is important in the monitoring of roads and streets. If the DMP were simply a flat plate, it would be too easy for the rod person to set up on a slightly different spot with each survey, especially if the monitored support element were bending inward, and this could result in cumulative errors in the elevation data plots. For support elements it is desirable that elevation surveys be carried out to an accuracy of as little as 1/4 or even 1/16 inch, and every effort should be expended to make this as easy for the surveyors as possible. The largest problem for this type of monitoring is the same as was previously discussed in ensuring survey accuracy, except that the difficulties may be greater in this instance because the surveyors are more likely to be working in the middle of heavy construction activity, hence more rushed and/or more distracted.

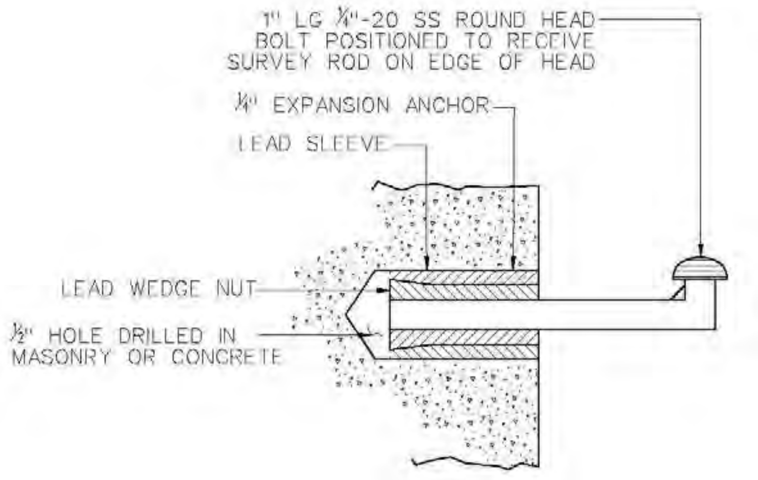


Figure 15-25 Deformation Monitoring Point in Vertical Masonry or Concrete Surface

15.4.2.2 Inclinometers in Slurry Walls

Inclinometers in slurry walls are very similar to those previously described for ground installations, except that drilling is not generally required (Figure 15-26). Installation is accomplished by fastening the instrument casing inside the wall panel's rebar cage as that element is being fabricated. As the cage is lowered into the slurry trench, the inclinometer casing goes with it and remains in place as the slurry is displaced during the introduction of concrete. Because the slurry wall will have been designed to penetrate below any zone of expected movement, the bottom of the inclinometer casing is the presumed unmoving reference from which tilting of shallower points along the casing are calculated. Monitoring is accomplished by the instrumentation specialist lowering a probe to the bottom of the casing and collecting readings as it is winched back to the surface. The biggest problem with an inclinometer in such an installation is the essential impossibility of repair if anything has gone seriously wrong. Also, one cannot replace the instrument by simply drilling a new casing into reinforced concrete a foot or two away. If the instrument is considered absolutely essential, it might be feasible to drill a new one into the ground just in back of the wall, but long drill holes tend to wander away from the vertical – perhaps in a direction away from the slurry wall – and chances are not good that the replacement instrument would truly indicate what the slurry wall itself is doing. This possibility of damage is one argument against the installation of in-place inclinometers in this type of support. Depending on the seriousness and the depth of any damage to the casing, some or most of the expensive sensors could be stuck and impossible to recover.

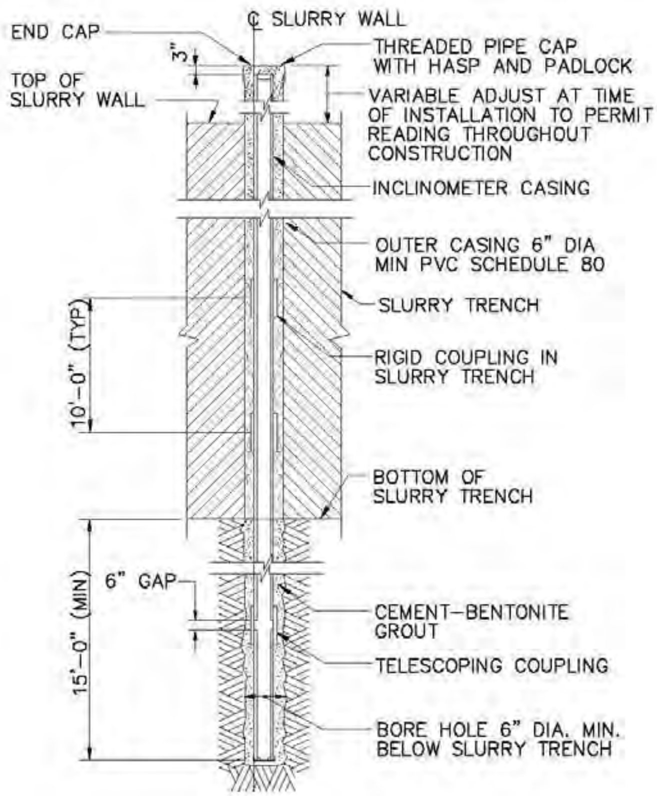


Figure 15-26 Inclinometer Casing in Slurry Wall

15.4.2.3 Surface Mounted Strain Gages

Surface Mounted Strain Gages are most commonly used to determine stresses and loads in struts across braced excavations. Although many kinds are available, the vibrating wire type finds the widest application because of a stable output that is in the form of signal frequency rather than magnitude. Figure 15-27 shows a schematic of the vibrating wire type strain gage. In this instrument's packaging, a length of steel wire is clamped at its ends inside a small housing and tensioned so that it is free to vibrate at its natural frequency. The frequency varies with the tension, which depends upon the amount of compression or extension of the instrumented strut to which the gage has been attached by spot welding or bolting. The wire is magnetically plucked by a readout device, and the frequency changes measured and translated into strain, which can in turn be translated into stresses and loads on the instrumented member from a knowledge of the material's modulus. The point of the measurements is that designers will have calculated the permissible loads in the struts and the instrumentation specialist is collecting data to determine if the struts may be approaching their design limits. Gages are typically mounted 2 to 3 strut widths/diameters from the ends in order to avoid the "end effects" that degrade accuracy. Because a strut will bend downward from forces of gravity even when not under load, creating compression at the top and extension at the bottom, it is necessary to install several gages arranged in patterns around the neutral axis and average the readings for the closest possible approximation of maximum stress.

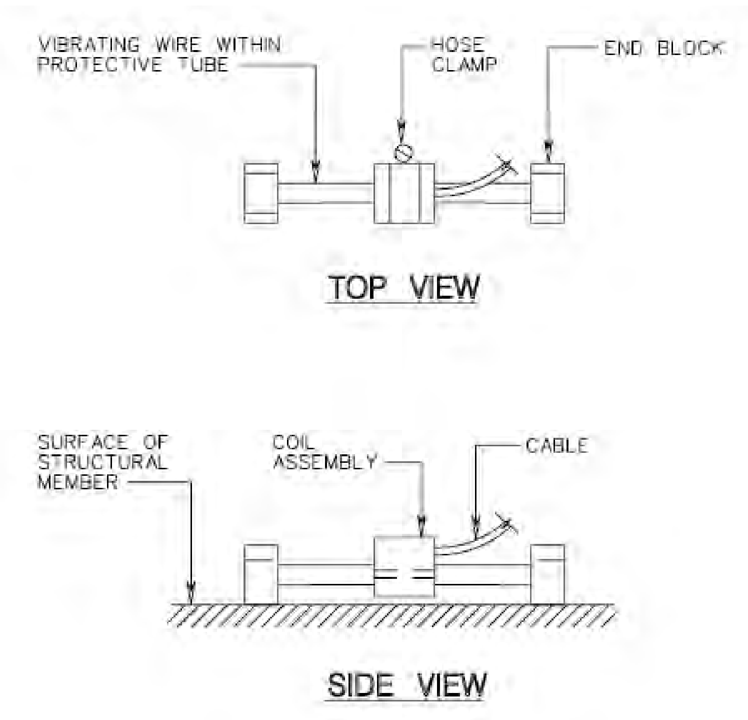


Figure 15-27 Surface Mounted Vibrating Wire Strain Gauge

Many things can go wrong with such installations, and they need to be undertaken with the greatest of care by experts with good experience. However, as noted in the introduction, the greatest problem with these types of measurements can reside in the agendas of the various parties who may need to understand the data and perhaps take action to mitigate apparent problems. Measurements of ground and structure *movements* are in general understood by most people associated with tunneling. However, stresses and strains require a certain amount of sophistication to comprehend, and even among those with the sophistication, interpretations of what the data mean can vary wildly. It is very common for constructors and their consultants to believe instruments are faulty, that data has not been properly collected, or data has not been properly reduced to good engineering values if taking mitigative action is going to interfere with the field operations. Also as previously noted, this is why use of strain gages can be fraught with complications if used on the steel ribs in mined tunnels. Compared with struts in braced excavations, ribs under load can bend and twist in many unanticipated ways, and placing strain gages in the best configurations just where they need to be placed can be difficult.

15.4.2.4 Load Cells

Load Cells are, in general, arrays of strain gages embedded in housings which are placed in instrumented tunnels under construction in such a way that loading forces pass through the cells. For the reasons stated in the strain gage description above, very stable vibrating wire transducers are the data collecting elements on which most load cell configurations are based. As shown in Figure 15-28, the load cell is a "donut" of steel or aluminum with several transducers mounted inside in a way to be read separately and averaged in the readout device. Transducers are oriented so that half of them measure tangential strains and half of them measure axial strains. Integration of the individual strain outputs helps reduce errors that might result from load misalignment or off center loading. Although load cells may be installed on tensioned rockbolts in mined tunnels, their more common use is in non-braced open excavations. Here the cell is installed on a tieback near the rock face and locked down with thick bearing plates, washers and a large steel nut. In most cases the instrument will be wired for electrical remote reading because it will be left in place for a considerable amount of time, and direct access for data collection will often not be available once the excavation has passed below the tieback's level. If a load cell seems to be producing questionable data, the most likely cause is misalignment of the instrument on the shaft of the tieback. For the most part, tiebacks are angled downward rather than being installed horizontally, and careful placement of bearing plates and washers of the correct thickness is essential.

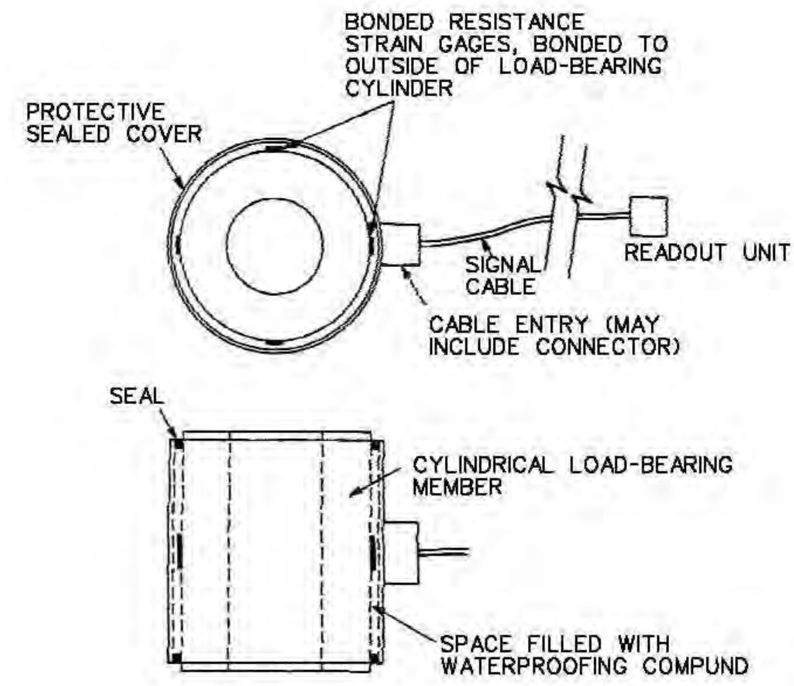


Figure 15-28 Schematic of Electrical Resistance Load Cell (After Dunnicliff, 1988, 1993)

15.4.2.5 Convergence Gages

Convergence Gages may be used on tunnel supports just as they are in monitoring of tunneled ground as described in 15.2.2. above. For the most part it is best to monitor the ground itself because that gives the best from-the-beginning measurements that constitute good initial movement readings. However, if it is necessary for whatever reason, similar anchors, eyelets, cradles and survey targets can also be installed on steel supports, shotcrete linings, and final concrete linings. As in the earlier discussion, it appears that distometers should be the chosen replacement for the older tape extensometers when measuring the distortions.

In modern mining there are situations which do not lend themselves to easy measurement of ground movements from the tunnel itself because of the chosen method of ground support. The most common of these situations results from the use of a TBM where pre-cast concrete segments are erected after each push to form another 4 or 5 feet of completed tunnel ring directly behind the shield. These theoretically perfect circles can distort as ground loads or other pressures – as from a contiguous tunnel also under construction – begin to exert themselves. The tunnel lining may "oval" with long axis vertical from high side pressures, or oval with long axis horizontal from high vertical pressures or low side pressures (the contiguous tunnel again.) Most instrumentation specifications call for deformation measurements to begin as soon as possible and for them to be taken as often as once or twice per day at first, with monitoring schedules tapering off as the TBM recedes from individual measurement sections. As with monitoring of ground movements, the most common problem with these measurements of lining distortion is the difficulty of getting good lines of sight directly behind the machine in order to achieve a true zero movement initial reading.

15.4.2.6 Robotic Total Stations

Robotic Total Stations as described for existing structures in 15.3.2. above can also be used to monitor the opening that is under construction. However, there are possibly more limitations on underground installations than on installations associated with inhabited buildings above. A total station instrument sitting atop its motorized support platform has a footprint of at least one square foot, its height is a bit greater, and the platform may protrude from the tunnel wall as much as 18 inches. The package would hardly fit well into a small tunnel, and would be constantly on the move as the tunnel advanced. Hence, the most logical place for such monitoring of active construction would be within a large mined chamber or perhaps a large open excavation. Even here, however, the uses might be more restricted than is at first obvious. The average construction site is a hostile environment, and the decision to install such an expensive piece of equipment cannot be taken lightly. The dust alone on some construction sites might be enough to force heavy maintenance procedures on the part of users. Even in the outdoors, target prisms have to undergo regular maintenance because signals can be so degraded by the accumulating dust from the atmosphere. The interior of a construction site is much worse; maintenance of the expensive instrument itself would be more onerous than usual, and many target prisms would likely be at a height that requires use of a manlift for access. It seems probable that the best use for robotic total stations would be found in an advanced stage of large construction where most of the final concreting has been accomplished and the structure needs to be monitored in something close to real time as the finish stage of construction proceeds.

15.5 DYNAMIC GROUND MOVEMENT – VIBRATIONS

15.5.1 Purpose of Monitoring

As opposed to the measurements discussed earlier, which concerned long-term effects of the construction of a tunnel on the gross movement of either the ground or buildings adjacent to the tunnel, these measurements are taken to establish the potential impact of drill and blast excavation on structures. Use of explosives often causes concern on the part of stakeholders in the neighborhood of a tunnel excavation. Aside from the images generated by blasting, there is real concern due to the sudden (and sometimes perceptible) motion generated by the explosive energy that is not used in fragmenting rock, but that propagates away from the blast site.

The usual method of monitoring these motions is based upon research studies that correlate the potential for damage from blast vibrations with the motion of the ground

15.5.2 Equipment, Applications, Limitations

There are two general types of equipment used for monitoring the Dynamic Ground Movement induced by blasting:

- Blast Seismographs

- Dynamic Strain Gages

Blast seismographs are used to monitor ground motion at structures within the zone of influence. Dynamic strain gages are used to monitor the actual strain (or relative displacement) of structural elements of such structures. Both of these instruments monitor data during the actual blast event, though for convenience they may be set to monitor before the actual blasting.

15.5.2.1 Blast Seismographs

The standard blast monitoring equipment has been blast seismographs. These instruments measure the vibration waves generated by blasting then propagate through ground, soil, and structures. This is the dynamic measurement of a wave that is extended in time and space; therefore, there is no single value that totally describes a blast wave. Through many years of research, it has been determined that the single most descriptive value that can be associated with the potential for structural damage is "Peak Particle Velocity," or PPV. As a blast vibration wave travels, it is analogous to waves on water. If one imagines a bobber on the water, the velocity of the bobber moving as the wave passes is the particle velocity. The peak particle velocity is the highest value of velocity during that wave passage. This value is expressed (in the US) in inches per second.

Blast seismographs measure three components of ground motion: vertical, longitudinal (horizontal along the direction from the blast) and transverse (perpendicular to that direction). The highest of these three values is used as a vibration criterion. There is typically a fourth channel used for above-ground blasting that monitors air overpressure or airblast, but this channel is generally not used when blasting in tunnels, since there is no direct exposure to surface structures.

As mentioned, criteria for blasting have been developed based upon occurrences of damage. Most of the studies done have concentrated on typical residential wood frame structures. Because structures respond in many ways to vibrations that are imposed at the base of the structure, in most cases the vibration is monitored on the ground outside of the structures. The potential for damage is then inferred from the association of the PPV with the potential for damage of a particular structure type. Sometimes the frequency of the vibration is also incorporated in the criteria, but this is not always the case. Criteria are usually adjusted upwards when the structure type is more substantial or engineered, relative to the criteria used for residential structures.

15.5.2.2 Dynamic Strain Gages

Because there is so little accumulated damage data for some structures, an alternative method for monitoring, using dynamic strain gages, has been adopted recently. For engineered structures and infrastructure elements, actual failure criteria can be developed that are independent of the mode of excitation. In this case, a level of strain, which is a dimensionless measure of relative motion, is used as a criterion for avoidance of damage. Strain ε is defined as $\varepsilon = \Delta l / l$, where Δl is the change in length of an element, and this is divided by the length of the element. Measurement on a small length of a structural element may then represent the deformation of the entire element when the total structural configuration is known.

Dynamic strain gages are traditionally thin foil resistance gages, which are connected to other gages in what is called a Wheatstone bridge. The gages change resistance when they are deformed. This arrangement of gages will then produce a voltage output that is monitored during the blasting process. The foil gages have been in use for over a half a century, initially in static strain environments, such as those described in 15.4.2.3 above. Though it is a mature technology, there are sometimes problems when the gages are in electrically noisy environments, or where there are temperature fluctuations. Although they have only been used recently, piezoelectric and fiber optic strain gages are not susceptible to as many problems as are the foil gages.

Dynamic strain gages, since they measure strain on a particular element that is of concern, must be carefully located to obtain the values that can be associated with potential failure of the element. Strain gage mounting must be carefully chosen on a representative location, and a measurement on the ground surface (as is done with blast seismographs) is NOT appropriate.

There is not as much background documentation in associating damage with strain from blasting; however the fundamentals of strain-based failure criteria have been used for many years. The use of strain gages is limited to where there is a sound understanding of the actual limiting strain values that can be accepted as safe, based upon engineering documentation.

15.6 GROUNDWATER BEHAVIOR

15.6.1 Purpose of Monitoring

In a landmark 1984 study titled *Geotechnical Site Investigations for Underground Projects*, the National Academy of Sciences catalogued problems associated with the construction of 84 mined tunnels in the U.S. and Canada, and stated bluntly in its conclusion, "The presence of water accounts, either directly or indirectly, for the majority of construction problems." Thus, even if groundwater does not flow into an advancing excavation in huge quantities to become a primary problem, it may still alter the ground in a way to make its behavior worse than it would otherwise be, and so become a serious secondary problem. For example, seemingly solid rock may be destabilized by the presence of water if the liquid carries binding particles out of otherwise closed joints or lubricates the joint faces to decrease frictional forces that hold rock blocks in place. Soft ground fares even worse in the presence of water as seepage forces may carry materials into the excavation, thus exacerbating the loss of ground, or perhaps causing subsidence above simply due to the pumping of water if the overlying soils are compressible. Most tunneling experts know that somewhat controllable "running ground" may become much-harder-to-control "flowing ground" if water is present and its effects are not checked. It is a given that, in most soft ground mined or cut-and-cover excavations where the water table is high, some kind of dewatering will need to be carried out to keep the headings safe. It is also a given that, even if formal pre-construction dewatering is not carried out, the excavation will probably cause a decrease in the level of the groundwater as intruding water is pumped out to create dry, workable conditions. Interestingly, even the drying up of the ground to make tunneling easier can have its own unwanted side effects if there are abutting facilities that depend upon the water table staying close to its original elevation for them to maintain their functionality.

15.6.2 Equipment, Applications, Limitations

Three standard types of instrumentation are used to determine the effect of tunnel construction on groundwater movements and pressures:

- Observation Wells
- Open Standpipe Piezometers
- Diaphragm Piezometers

15.6.2.1 Observation Wells

Observation Wells are the simplest and least expensive instruments in the list of devices used to determine groundwater pressures. A well consists of a perforated section of pipe attached to a riser pipe installed in a borehole filled with filter material, generally sand or pea gravel (Figure 15-29). The filter prevents fines from migrating in with the water and clogging the well. The filter may extend to only a few feet above the perforated section or may go almost to the ground surface, but the well must have a mortar seal near the top of the riser pipe to prevent surface runoff from entering the hole. Also, a vent is required in the top cap so that water is free to rise and fall in the pipe. The height of the groundwater table is generally measured by lowering an electrical probe at the end of a graduated cable until it touches the top of the water. A circuit is then completed and so indicated by the flicker of an indicator light or sound of a buzzer at the upper end of the cable. Such wells are installed in tunneled ground where it is assumed that the ground is continuously permeable and groundwater pressures will increase uniformly with depth. Tunnel designers try to gain an understanding of the groundwater regime as design proceeds and often will specify the level to which the water must be pulled down by a dewatering program before construction is permitted to proceed too far. It is common to require dewatering to a level a few feet below final invert for either a soft ground mined tunnel or braced excavation. An observation well would then be installed to two or three feet below that drawdown level to be certain of detecting the new during-construction top of water table. The most common problem with observation wells is that they may not be the instrument appropriate for the situation because the complexity of geologic stratification is actually greater than anticipated. If readings seem inexplicable, it may be because the water level corresponds to the head in the most permeable zone rather than to a straight line correlation with depth from the ground surface. It is possible that the wells may need to be supplemented with other instruments such as piezometers.

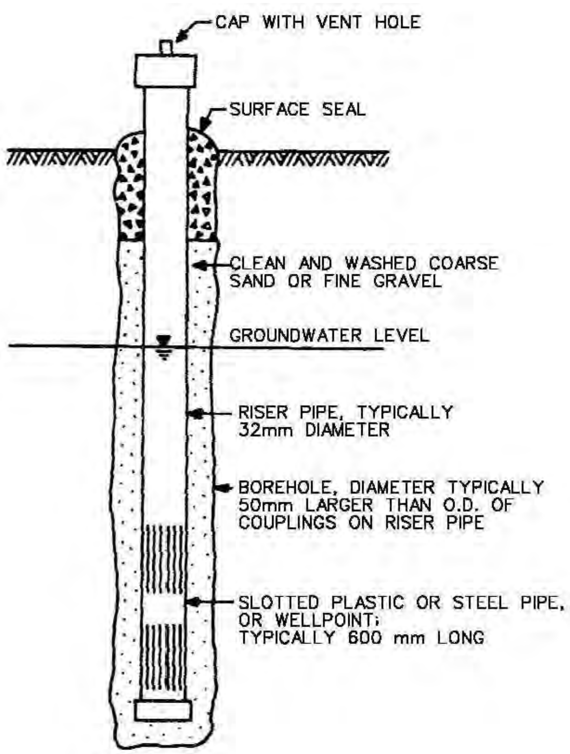


Figure 15-29 Schematic of Observation Well (After Dunnicliff, 1988, 1993)

15.6.2.2 Open Standpipe Piezometers

Open Standpipe Piezometers are very similar in construction to observation wells, with one major difference: as defined by Dunnicliff, the porous filter element is sealed with bentonitic grout into a particular permeable stratum so the instrument responds to groundwater pressure only at that level and not to pressures at other elevations (Figure 15-30). Such a piezometer may be installed in soil strata or in bedrock and will function as long as the porus intake and filter are sealed in a zone that permits water to flow. In soil the instrument will be measuring pore water pressure; in rock, it will generally be measuring joint water pressure. The instrument creates little or no vertical hydraulic connection between strata and, in contrast to simple observation wells, readings will be more accurate. If stratification is somewhat complex, several piezometers installed at different depths in the same small area would probably reveal more than one level of pressures, as in the case of a perched water table above a clay stratum exhibiting pressures different from those in a permeable stratum below the layer of clay. In construction monitoring it is usual to install the porus intakes at the critical levels only, as in just below the inverts to where the water table needs to be lowered. Another common depth for the intakes would be at the boundary between an upper layer of sand and a lower layer of impervious clay in which the excavation bottoms out. In the latter situation, the dewatering subcontractor would probably be able to pull the water table down only to a few feet above the clay, and that is the elevation that would need to be monitored. Lack of expected response from an open standpipe piezometer is sometimes caused by clogging of the filter due to repeated water inflow and outflow. This may be remedied by high pressure flushing, something readily accomplished if the drill rig used during installation is still in the area. A more serious problem would result from the porous intake having been installed in a relatively impermeable silt or clay stratum because the borehole was not properly logged prior to installation. The only solution would probably be to install another instrument – perhaps another type of instrument – at the same plan location, with more attention being paid to good geologic logging and placement of the porous intake.

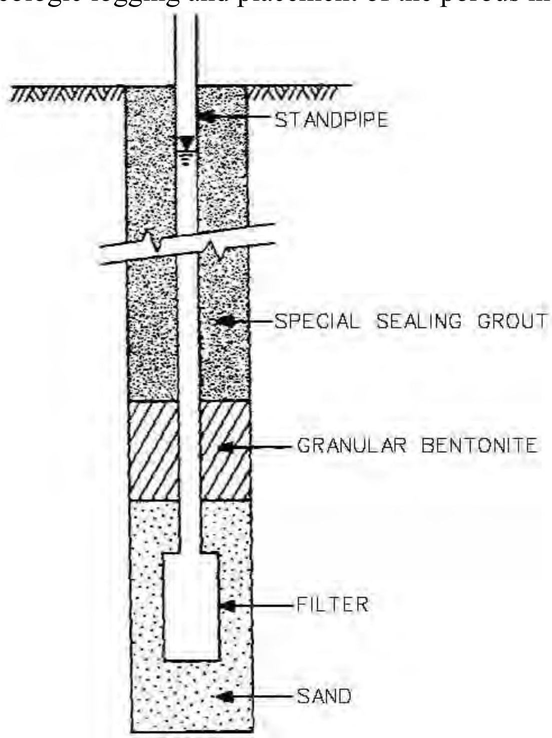


Figure 15-30 Schematic of Open Standpipe Piezometer Installed in Borehole (After Dunnicliff, 1988, 1993)

15.6.2.3 Diaphragm Piezometers – Fully Grouted Type

As noted earlier, a *piezometer* is a device that is sealed within the ground so that it responds only to groundwater pressure around itself and not to groundwater pressures at other elevations. There are several situations that point to the need for a device that is more sophisticated than the simple open standpipe instrument:

1. Need to measure pore water or joint water pressure in a stratum of very low permeability. The hydrodynamic time lag for an open standpipe instrument is large, meaning that it responds slowly to changes in piezometric head because a significant volume of water must flow to register a change. This cannot happen in materials of low permeability such as clay or massive bedrock with few joints.
2. Some situations make it undesirable to have a rigid standpipe connecting with the surface, especially in the midst of heavy construction.
3. Repeated water flow reversals can cause the sand or pea gravel filter to clog.
4. In very cold climates there is a chance of freeze-up and resultant loss of opportunity to collect data.
5. A large number of readings and/or something close to real time monitoring may be required, but the open standpipe instrument does not lend itself readily to this type of data collection.

Thus there are times when monitoring personnel are forced to choose a type of piezometer consisting of a unit that is pre-manufactured to interpose a diaphragm between the transducer and the pressure source. Pneumatic, electrical resistance and vibrating wire are the three most common type of such instrument. The vibrating wire type us usually chosen because it operates with a short time lag, offers little interference to construction, and the lead wires can easily be connected to a surface readout unit or to a datalogger for real time monitoring.

Even these instruments, however, have always suffered from a major shortcoming: the assumed need to place filters around the sensing units and granular bentonite/cement grout seals and backfilling in the boreholes around and above the monitored elevation. Bridging and material stickiness can make proper emplacement difficult and may lead to degradation of data accuracy or outright instrument malfunction. This emplacement difficulty particularly complicates the installation of multiple piezometers in one borehole, so if readings from various elevations are required, it may mandate the drilling of a separate hole for each elevation that requires measurement.

An obvious way around these difficulties would seemingly have been to forgo the filters and encase diaphragm piezometers and their accoutrements in a cement-bentonite mix seal all the way to the surface in *fully-grouted* installations. However, prevailing opinion for many years was that the grout around the sensing unit might have extremes of permeability that would prevent an instrument from responding accurately to changes in pressures. But from work that began in 1990, it has now been shown that this does not have to be the case. A diaphragm piezometer generally requires only a small flow to respond to water pressure changes, and the grout is able to transmit this small volume over the short distance that separates the sensing unit from the ground in a standard size borehole. The response can be enhanced if the installer minimizes this distance, which can be accomplished through the use of an expandable assembly that lessens the distance between sensor and borehole wall, thus reducing the thickness of the grout between sensor and ground. Studies have shown that accuracy of pressure measurements will be good not only when the permeability of the grout is lower than that of the surrounding ground (which had been assumed all along), but also when the permeability of the mix is up to three orders of magnitude greater than that of the surrounding ground. Obviously, every situation requires that some work be done to formulate a grout mix of an appropriate permeability to be effective at the site being monitored.

Fully-grouted piezometers can be emplaced by loose attachment then detachment from a sacrificial plastic pipe that is withdrawn (along with any support casing) as the grout is tremied in from the bottom up. It is relatively easy to install more than one instrument in the same hole for water pressure measurements at several elevations. As many as ten in holes penetrating to 500 ft depths have been successfully installed.

Good experience in a greater than 15-year time frame prior to 2009 has shown that most diaphragm piezometers need to be installed as fully-grouted types for the sake of increased simplicity and the collection of much more data at lower cost than had been the case with older methods.

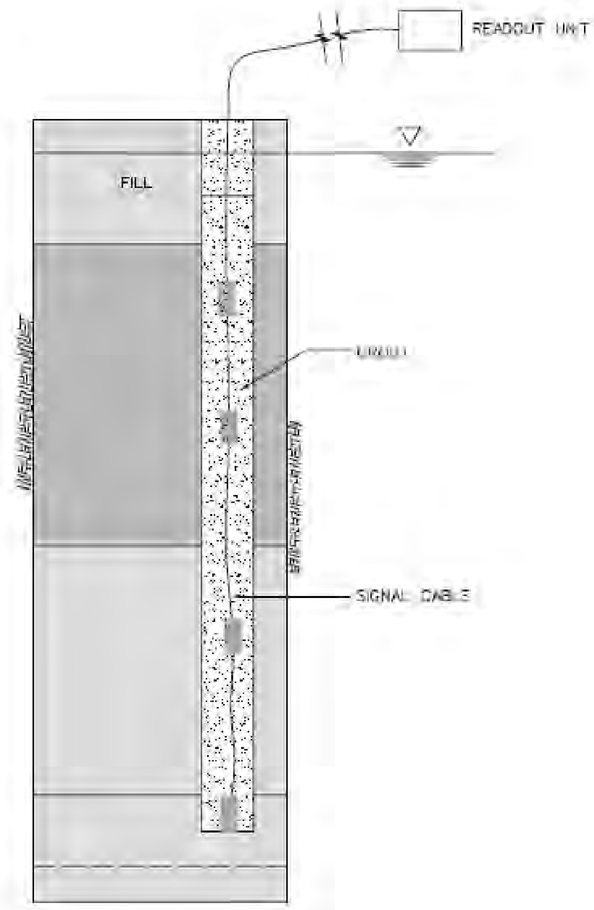


Figure 15-31 Schematic of Multiple Fully-Grouted Diaphragm Piezometer

A continuing use for piezometers and observation wells depends upon their being left in place after construction is complete because of the effects the permanent structure may have on the groundwater regime. For example, if the water table remains depressed due to leakage into the new tunnels, a continuation in monitoring may indicate whether attention needs to be paid to protection of wood support piles that remain exposed to air, or perhaps to wells or ponds that have been wholly or partially dried up. An opposite problem may stem from the mounding up of groundwater because it's normal gradient is interrupted by the presence of the new tunnel, which may result in situations such as once dry basements that are now prone to flooding. Although leaving the instruments in place may result in increased maintenance costs, they can prove to be valuable sources of data when certain long term problems are being investigated.

15.7 INSTRUMENTATION MANAGEMENT

15.7.1 Objectives

As noted in the introduction to this chapter, the primary function of most instrumentation programs is to monitor performance of the construction process in order to avoid or mitigate problems. There are, of course, other related purposes, and proper management of the program will include decisions on which of the following deserve primary consideration and which may be considered of lesser importance:

1. To prevent or minimize damage to existing structures and the structure under construction by providing data to determine the source and magnitude of ground movements.
2. To assess the safety of all works by comparing the observed response of ground and structures with the predicted response and allowable deformations of disturbance levels.
3. To develop protective and preventive measures for existing and new structures.
4. To select appropriate remedial measures where required.
5. To evaluate critical design assumptions where significant uncertainty exists.
6. To determine adequacy of the Contractor's methods, procedures and equipment.
7. To monitor the effectiveness of protective, remedial and mitigative measures.
8. To assess the Contractor's performance, Contractor-initiated design changes, change orders, changed conditions and disputes.
9. To provide feedback to the Contractor on its performance.
10. To provide documentation for assessing damages sustained to adjacent structures allegedly resulting from ground deformations and other construction related activities.
11. To advance the state of the art by providing performance data to help improve future designs.

An overriding factor in considering what is important about instrumentation may spring from new demands being made by insurance and bonding companies. In many parts of Europe they already have the power to require that every tunneling project, prior to construction, undergo a process of *Risk Analysis* or *Risk Assessment*. Then, during construction, periodic audits are conducted to determine whether a project is successfully practicing *Risk Management*. A low score on this point could result in the cancellation of insurance and the possible termination of the project. Although not yet to such an advanced stage, the tunneling industry in the U.S. is becoming very attuned to the necessity of Risk Analysis and Management, and a good instrumentation program can help to reduce the possibility of major problems. It can be shown to the satisfaction of most observers that a good monitoring program has the potential to pay for itself many times over through the monies saved from incidents that were prevented from happening. In other words, Risk Management backed up by good instrumentation and monitoring can be very cost effective.

15.7.2 Planning of the Program

Much of the following material is predicated on the assumption that any particular project will follow the standard U.S. Design-Bid-Build method of services procurement. Where an alternative method such as Design-Build may be a possibility, we will try to point out how this could affect the instrumentation program under consideration.

As noted by Dunnicliff (1988, 1993), the steps in planning an instrumentation program should proceed in the following order:

1. Predict mechanisms that control behavior of the tunneling medium
2. Define the geotechnical questions that need to be answered

3. Define the purpose of the instrumentation
4. Select the parameters to be monitored
5. Predict magnitudes of change
6. Devise remedial action
7. Assign tasks for design, construction, and operation phases
8. Select Instruments
9. Select instrument locations
10. Plan recording of factors that may influence measured data
11. Establish procedures for ensuring reading correctness
12. List the specific purpose of each instrument
13. Prepare a budget
14. Write instrument procurement specifications
15. Plan installation
16. Plan regular calibration and maintenance
17. Plan data collection, processing, presentation, interpretation, reporting, and implementation
18. Write contractual arrangements for field instrumentation services
19. Update budget

Many of these points will be covered in more detail in the following pages, but no. 2 deserves special emphasis here; Dunnicliff stated it in the following terms:

> ***Every instrument on a project should be selected and placed to assist in answering a specific question: if there is no question, there should be no instrumentation.***

The basic point can also be stated as, "Do not do something just because it is possible or because it might result in something that would be nice to know." Movement in that direction can result in wasted monies and the proliferation of excess – perhaps even conflicting – data that leads to confusion.

Serious work on planning an instrumentation program will probably not begin until some time after the 30-percent design level has been completed because only then will such aspects of the project as geology, tunnel alignment, structural design and probable methods of construction be coming into good focus. Program design should be carried out by geotechnical engineers and geologists who have a good knowledge of instrumentation, assisted as necessary by the structural engineers with the most knowledge of how the new and existing structures are likely to react to the changing forces to which they will be subjected.

15.7.3 Guidelines for Selection of Instrument Types, Numbers, Locations

Due to the large number of permutations and combinations of highway tunnel types, sizes, depths and geographic/geologic locales, it would be very difficult to list truly useful guidelines in the space allotted herein. A few of the authors' thoughts on the subject can be found in preceding sections 15.3 through 15.6, but even those 20 or so pages can only begin to suggest what can or should be done. But in addition to space limitations, there is also a danger in the listing of specific guidelines in a manual such as this because it can lead to a user's thinking of the materials as a "cookbook" in which the solutions to most problems are contained and for which no further thought needs to be given. Instrumentation and monitoring is too large a subject for this kind of treatment, and readers are urged to absorb the contents of as many of the listed references as possible in order to knowledgably compile their own project-specific guidelines for the undertaking at hand. That suggested task is summarized in nos. 8 and 9 in section 15.7.2. above.

15.7.4 Remote (Automated) versus Manual Monitoring

As noted in the introduction, the automation of many, perhaps most, types of instrumentation is now possible and in some cases even relatively easy. This does not mean that it should always be done because increasing sophistication may also mean an increase in front end costs, maintenance costs, and in the number of things that can go wrong. Some of these considerations were covered briefly in preceding paragraphs, but without any large generalizations or guidelines having been promulgated.

It is easy to lose sight of one of the advantages of hands-on, manual monitoring, namely that it puts the data collecting technician or engineer on the job site where he or she can observe the construction operations that are influencing the readings. This can be a huge advantage because the interpretation of instrumentation data requires the comparison of one instrument type with another for mutual confirmation of correctness, and then seeing if the data plots match up with known construction activities, such as the removal of a strut or the increased depth of an excavation. Without such information being provided by the geotech field personnel, the instrumentation interpreter has to spend time digging out construction inspectors' reports or talking with various other people who may have knowledge of daily occurrences at the site. Valuable time can thus be lost, a serious consideration if adverse circumstances are developing fast. However, if data interpreters are depending upon their field personnel to provide feedback, those personnel need to have at least some minimal training in construction terminology and methods. For example, it is not helpful if monitoring personnel do not have the vocabulary to note whether they are observing the installation of a strut or a whaler.

Following are some of the most important reasons for choosing automation over manual monitoring of instruments:

1. When there is a requirement for data to be available in real time or something close to real time.
2. When easy and/or continued access to a monitored location is not assured.
3. When there is uncertainty about the continued availability of monitoring personnel.
4. When manual readings are subject to "operator sensitivity" and the same person or crew cannot always be available to monitor an instrument time after time.
5. When manual monitoring would unduly interfere with construction operations.
6. When manual monitoring would be too time consuming; e.g., the several-times-per-day reading of conventional inclinometers.
7. When data needs to be turned around quickly and distributed to multiple parties located in different offices.

15.7.5 Establishment of Warning/Action Levels

At one time it was common for instrumentation program designers to write specifications on equipment types and installation procedures, but then leave up to construction contractors and field instrumentation specialists the decisions on whether allowable movements (or other parameters) were about to be exceeded. This can lead to endless arguments on whether mitigative action needs to be taken and whether the Contractor deserves extra payment for directed actions he may not have forseen when submitting a bid price. Such problems can be alleviated to a degree by specifying the instrument reading levels which call for some action to be taken. Depending on a project Owner's preferred wording, the action triggering levels may be called instrument *Response Levels*, comprised of *Review* and *Alert Levels*, or *Response Values*, comprised of *Threshold* and *Limiting Values*.

The actions are generally specified in the following manner:

A. If a Review Level/Threshold Value is reached, the Contractor is to meet with the Construction Manager to discuss response actions. If the CM so decides, the Contractor is to submit a plan of action and follow up within a given time frame so that the Alert Level/Limiting Value is not reached. The CM may also call for the installation of additional instruments.

B. If, in spite of all efforts, the Alert Level/Limiting Value is reached, the Contractor is to stop work and again meet with the CM. If the CM so decides, the Contractor is to submit another plan of action and follow up within a given time frame so that the Alert Level/Limiting Value is not exceeded. Again, the CM may also call for the installation of additional instruments.

Such wordsmithing is easy compared with the effort involved in actually deciding what kind of levels/values to specify, because it may entail much time spent in structural and geotechnical analysis. It is not uncommon for specifications to stipulate only the actions required when settlements of any existing structure have reached a certain magnitude, or when the vibrations from blasting have exceeded a certain peak particle velocity. However, there are many other parameters that may deserve attention. Following is a partial list of what may be appropriate to consider for inclusion in specifications:

- Depth to which groundwater level must be lowered or depth to which it may be permitted to rise.
- Allowable vertical movements of anchors or sensors located at various depths in the ground.
- Allowable lateral deflections from the vertical as stated in relation to the depth of any sensing point in an inclinometer.
- Allowable deformations of ground or linings in the tunnel under construction.
- Allowable settlements for individual existing structures (as opposed to one set of figures applying to all structures equally).
- Allowable tilting of the walls in individual existing structures.
- Allowable differential settlements and angular distortions for existing structures.
- Allowable increases in widths of structural cracks or expansion joints.
- Allowable load increases in braced excavation struts or tiebacks in non-braced excavations.
- Rate of change of any of the above, in addition to the absolute magnitude.

In the interest of good risk management, it is recommended that designers of instrumentation and monitoring programs include what they consider the most important of the parameters in the specified action-triggering levels.

As these levels are being set, designers should guard against one pitfall: the assignment of readings that are beyond the sensing capabilities of the instrument. For instance, if a lower action-triggering level of ¼ inch has been specified for a settlement point, one must be assured that the survey procedures used to collect data can reliably detect settlements down to 1/16 inch, for otherwise construction managers may be constantly responding to apparent emergencies that are not real but are only a result of survey "flutter." Likewise, the higher action-triggering levels must be set a realistic distance above the lower ones to avoid similar problems. In the noted example, a lower level of ¼ inch perhaps should not be matched with an upper level of 3/8 inch because that is an increase of only 1/16 inch, still pushing the level of probable surveying accuracy. Again one might end up responding to apparent emergencies that are not real.

15.7.5.1 Criteria

It is not within the scope of this document to establish criteria for tunneling projects; however, any monitoring program that is developed to protect adjacent properties must be consistent with both the types of measurements as well as the actual limiting values that are consistent with standard industry practice.

Criteria may be set either by regulations (Federal, State, and/or Local), or by specifications.

Measurement Category	Instrumentation	Type of Reading	Units
Ground Movement	Survey Point	Displacement	Inches
Dynamic Ground Movement	Blast Seismograph	Peak Particle Velocity	Inches/second
Dynamic Ground Movement	Strain Gage	Strain	Microstrain

15.7.6 Division of Responsibility

15.7.6.1 Tasks or Actions

Tasks or Actions required for an instrumentation and monitoring program can be summarized as follows:

1. Lay out, design, specify.
2. Procure/furnish.
3. Interface with abutters for permission to install.
4. Install.
5. Maintain.
6. Monitor.
7. Reduce data.
8. Maintain database.
9. Distribute reduced data.
10. Interpret/analyze data.
11. Take mitigative action as required.
12. Remove instruments when need for them is ended.

<u>Potential Performing Entities</u> include the following four, any of whom may be assisted by a specialist consultant or subcontractor:

- The Owner
- The Design Engineer (not a separate entity in cases where the state – the Owner – is also the designer)
- The Construction Manager
- The Construction Contractor

In the case of Design-Build contracting, it is essentially a given that the Construction Contractor will be responsible for all of the listed tasks. This entity will probably be assisted by a consulting engineering firm to carry out the general design, and by an instrumentation specialist to attend to the matters related to instrument procurement, installation and monitoring, but it is the Contractor who takes the overall responsibility for the project.

In the more general (for the U.S.) case of Design-Bid-Build contracting, decisions have to be made by the Owner on how to assign the various responsibilities. Ideally, the Owner or the Owner's designer or

Construction Manager should be responsible for all of the 12 listed tasks except for nos. 3 and 11. Since the Contractor is not even aboard at the time of instrumentation program development, the tasks related to no.1 have to be undertaken by the designers of the project. The Contractor could perform no. 3 and must be the one to perform no. 11. (More will be said shortly about task no. 10.)

In the real world, it is a fact that most owners prefer to relegate to contractors the responsibility for furnishing, installing, maintaining and removing instrumentation, often because it, as a result of being included in a competitive low bid process, seems to provide equipment and services at the lowest possible cost. However, monies that seem to be saved by this decision may be less than they at first appear because low-bidding contractors will seldom opt for the highest quality instruments and will probably be constantly pushing for alternative instrument types for their own convenience rather than for the good of the project. Such contractor responsibilities can be considered acceptable only if the following rules are adhered to: (a) specifications must require the services of properly qualified instrumentation specialists; (b) specifications must be very detailed in the requirements for instrumentation hardware and installation methods, especially if the project is broken up into multiple contracts, where consistency from contract to contract has to be assured; and (c) the CM's staff must make every effort to diligently review contractor submittals and to inspect the field work as installations proceed.

If these rules are followed, it may be acceptable to turn over tasks 2, 4, 5 and 12 to a construction contractor, but one thing must be borne in mind: the Contractor's primary job is to *construct*. Instrumentation related activities are peripheral to that job; they will probably be viewed by the Contractor as a nuisance at best, and possibly as deleterious to progress. The CM needs to be cognizent of this attitude and thus to exercise the oversight necessary to ensure that unacceptable shortcuts are not taken.

One other aspect of low bid construction work can make relegation of these tasks to the constructor at least acceptable if not exactly desirable. When instrument installation is carried out by forces directly responsible to the Owner, there are many instances where the Contractor will have to provide assistance, perhaps even going so far as to shut down operations for a time. This can lead to endless friction with the CM and very likely to many claims for extras as the Contractor perceives too much interference in the construction process. Some of this conflict can be avoided if the actions of the instrument installation personnel are more under the control of the party responsible for progressing the primary job of excavation and support, i.e., the Contractor.

It can never be good policy, however, to turn the instrumentation monitoring, databasing, and data distribution over to the party whose actions are being "policed" through use of that data. Data collection and related tasks must be the responsibility of someone answering directly to the Owner, and that would normally be the Construction Manager. However, along with the responsibility for monitoring must go the responsibility, not just for distributing the reduced data, but also distributing it within a useful time frame. This normally means the morning after the day on which the data is collected, but in the modern world it may be much faster. With many instruments being monitored electronically in real time, and the data fed directly to the Project's main computer, much data can be delivered around the clock and alerts can be issued to users of cellphones and laptops when there is indication that action trigger levels have been reached or exceeded.

Regarding the interpretation of instrumentation data (task no. 10 above) the CM's forces will have to do it as a matter of course to ensure that construction operations are proceeding according to specification. However, it is not incumbent on the CM to immediately deliver interpretations to the Contractor along with the data. The Contractor is still the party with primary responsibility for safety of the job, and therefore, he must also have responsibility for performing an independent interpretation of what the monitoring data means and stand ready to pursue whatever mitigating actions seem indicated. Otherwise, the Owner will have bought into a part of the responsibility for safety that by right belongs elsewhere.

15.7.7 Instrumentation and Monitoring for SEM Tunneling

As discussed in Chapter 9, instrumentation and monitoring is an integral part of the SEM tunneling for the verification of design assumptions made regarding the interaction between the ground and initial support as a response to the excavation process by means of in-situ monitoring. It aims at a detailed and systematic measurement of deflection and stress of the initial lining. Monitoring data are collected thoroughly and systematically.

Readers are referred to Chapter 9 “SEM Tunneling” for discussions about monitoring management for SEM application.

CHAPTER 16
TUNNEL REHABILITATION

16.1 INTRODUCTION

This chapter focuses on the identification, characterization and repair of typical structural defects in a road tunnel system. The most significant problem in constructed tunnels is groundwater intrusion. The presence of water in a tunnel, especially if uncontrolled and excessive, accelerates corrosion and deterioration of the tunnel liner. This chapter identifies the methods for measuring the flow of water from a leak; describes proper methods for identifying the types of remedial action to be taken including sealing of the liner with either chemical or cementations grout; and describes the procedures to install the various types of grout. A comparison of types of grout available at the time of writing and a chart indicating which type of grout is best suited for each condition is provided. Typical details are included to illustrate the proper methods for grouting.

This chapter presents various structural repair methods to reinstate the structural capacity of a deteriorated tunnel liner including methods for demolition of unsound concrete, brick or steel and methods for the restoration of the tunnel liner to its original condition and function. Details for the repair of concrete, steel reinforcement, and embedded elements of the tunnel liner system are provided. Most of the repair methods presented are designed to be used in active tunnels which permit minimum daily shutdowns. Repairs can be performed in a limited time frame allowing the tunnel to be returned to service on a daily basis.

This Chapter also addresses the structural bonding of cracked concrete. Details are presented to illustrate methods for demolition, surface preparation and placement of concrete to complete repairs. Current state-of-the-art materials available for repair of cast-in-place and precast concrete, steel and cast iron linings are discussed. Special procedures required for the repair of each lining material are presented.

This Chapter also addresses the various methods for the repair of components of segmental liners, including gaskets, attachments and fasteners. Guidelines for the repair of each type of segmental lining are presented. Design details of tunnel segmental lining are discussed in Chapter 10. The repair of hangers for suspended ceilings is discussed.

Repairs of steel/cast iron components addressed hereafter include roof beams columns, knee braces etc. which are often subject to severe corrosion and often need to be upgraded, replaced or rehabilitated. This Chapter covers typical details required for the restoration of riveted sections, rolled steel beams and other specially fabricated steel and cast iron elements of a tunnel system, and includes details on surface preparation, coatings for corrosion protection and proper methods for fire protection of the steel /cast iron elements of a tunnel.

This Chapter also address repairs of brick, dimension (Ashlar) stone and concrete masonry elements that exist in many tunnel systems. Methods of evaluating the condition of the masonry elements and methods for the restoration of masonry elements include removal and replacement, repair of mortar joints and methods for repointing joints. Procedures for the support of masonry structures during rehabilitation are discussed.

Lastly, structural repairs of unlined rock tunnels are briefly discussed in this Chapter.

16.2 TUNNEL INSPECTION AND IDENTIFICATION

Tunnel inspection requires multi-disciplinary personnel familiar with various functional aspects of a tunnel including civil/structural, mechanical, electrical, drainage, and ventilation components, as well as some operational aspects such as signals, communication, fire-life safety and security components.

Recognizing that tunnel owners are not mandated to routinely inspect tunnels and that inspection methods vary among entities that inspect tunnels, the FHWA and the Federal Transit Administration developed guidelines for the inspection of tunnels in 2003 and updated them in 2005 known as ‘‘Highway and Rail Transit Tunnel Inspection Manual’’ available at www.fhwa.dot.gov/bridge/tunnel/inspectman00.cfm (FHWA, 2005a). Note that at the time of preparing this manual, the FHWA is proposing to create a regulation establishing National Tunnel Inspection Standards (NTIS) which would set minimum tunnel inspection standards that apply to all Federal-aid highway tunnels on public roads.

This Manual and Chapter focus on the civil/structural aspect and assumes tunnel inspection to be performed by experienced personnel who are familiar with the types of materials found in tunnels, have a basic understanding of the behavior of tunnel structural systems, have had experience in the inspection of transportation structures and are familiar with the FHWA Bridge Inspection Training Manual (FHWA-FD-91-015), and Highway Rail and Transit Tunnel Maintenance Rehabilitation Manual (FHWA-IF-05-017) (FHWA, 2005b). In addition to the information identified in the Bridge Inspection Training Manual, protocols are described herein that are applicable to the inspection of road tunnels. The following sub-sections discuss the standard parameters for inspection and documentation.

16.2.1 Inspection Parameter Selection

Inspection parameters are chosen based upon the preliminary inspection of the tunnel and the scope of work. Particular emphasis should be placed on determining the presence of special or unique structures that require the addition of special inspection parameters for inclusion in the project database.

16.2.2 Inspection Parameters

Standardized inspection parameters are necessary to speed the processing and evaluation of the observed data. The use of standardized coding of information, necessary for consistency of reporting, also helps to assure quality control by providing guidelines for inspection personnel and standardizing visual observations. The Deficiency and References Legends in Appendix H provides a recommended standard coding for cataloguing tunnel defects.

16.2.3 General Notes in Field Books

All general field inspection/repair notes, consisting of a chronology of events, must be kept in a bound field book. Each member of the field team must carry a bound field book at all times when on site. The information contained in the field book should include notes on safety issues and on discussions with contractors, operations personnel and other interested parties. Entries into the field book must be chronological by date and time, and consist of clear, concise and factual notification of events and appropriate sketches. Field records, notes and the inspection database shall be maintained in one location. Field books should be copied on a weekly basis to prevent loss of data.

Nowadays electronic notebooks and/or special laptop computers are often used to record field data and sketches digitally which can also include digital photographs and/or videos with date, time, and GPS location information embedded.

16.2.4 Field Notes

The three types of field notes required for effective inspection of roadway tunnels are:

- General notes in field books
- Documentation of defects on field data forms
- Documentation of defects by photographs/video.

16.2.5 Field Data Forms

Field data forms document the information required for a particular project. In general, these forms are developed for the project and are project specific. The forms provide a project standard for the tabulation of the data obtained from the inspection. This information is transmitted to the data management personnel for input into the project database.

16.2.6 Photographic Documentation

The documentation of tunnel defects is best supplemented by the use of a digital camera. Photographs should be taken of typical and atypical conditions. Additionally, the photographs should also be used as documentation for special or unique conditions.

Photographs must:

- Exhibit the project number, date, time, location, photographer and a general description of the item
- Be catalogued and stored in a systematic manner for future recall. Note: It is helpful to name all photo logs in a consistent manner that is outlined in a directory; i.e., using the structure number as a pre-fix to each individual photo file name.

It is essential to follow the photographic method of documentation referenced above. Instituting this method at the beginning of the project will prevent mislabeled or unlabeled data from being distributed or misinterpreted.

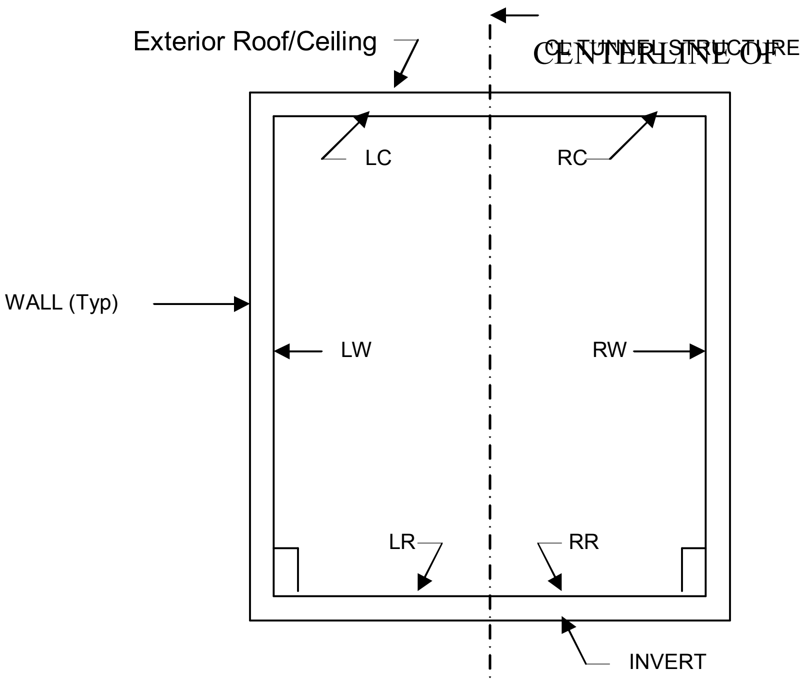
16.2.7 Survey Control

All condition surveys require a definitive baseline for location (survey) purposes. Generally most highway systems have an established survey baseline. The post construction baseline survey of the highway system is usually performed for the maintenance of the roadway and tunnel structure. Such stationing systems are usually well defined with permanent markers located on the tunnel walls. Some tunnels may already have a baseline condition established by laser scanning techniques (Chapter 3).

The tunnel inspection documentation must be linked to the existing baseline stationing system for the following reasons:

- Allow inspection data to be used for long-term monitoring of the tunnel structure by the owner's engineering/maintenance staff
- Allow defects to be readily located for future inspection or repairs
- Facilitate rapid start-up of inspection teams
- Reduce project costs and confusion

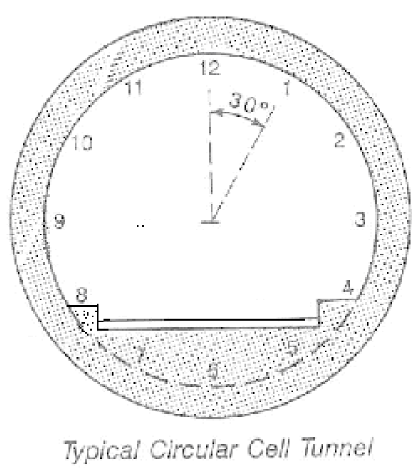
In addition to locating the tunnel defects along the alignment, it is necessary to locate them in relation to their position within the structure. To locate defects within the tunnel, the limits of the walls, roof, and invert must be delineated for conformity (Figure 16-1). Circular tunnels are divided up into 30-degree segments clockwise from the highpoint of the tunnel crown as shown in Figure 16-2. This delineation is always performed looking upstation on the established baseline survey.



TYPICAL CUT-AND COVER TUNNEL SECTION
(LOOKING UPSTATION)

Legend
LC&RC: Left& Right Ceiling ; LW&RW: Left & Right Wall; and LR&RR: Left & Right Roadway Wearing Course

Figure 16-1 Typical Cut and Cover Inspection Surfaces and Limits (Russell, 1992)



LOOKING UPSTATION

Figure 16-2 Delineation of Typical Circular Tunnel

The development of standard inspection parameters and the associated calibration of inspection crews, prevent many of the errors and omissions that can occur when the work is performed by numerous separate teams. In addition, timely reviews by the project advisory committee allow for program modifications and speedy implementation of supplemental procedures as required.

The documentation for each tunnel, boat section, ventilation building, cross passage, utility room, low point sump, pump station, air duct or other element is made looking up-station. The element being inspected is divided about the centerline. Each component of the element having deficiencies/ observations to the left of the centerline will have a prefix of (L), whereas those to the right of the centerline will have a prefix of (R).

Standardized codes are developed for deficiencies that correspond to each component of the tunnel structure. These deficiencies can be tracked easily in the field and conformed to by the inspection crew. Existing codes for deficiencies are depicted by symbols and identification for both concrete: spalls, delaminations, cracks and joints and steel: reinforcing and framing. Also identified are bolt connections, and tunnel moisture.

Spalls and delaminations may occur in concert and are almost always found in association with structural cracks. There are documented instances where spalls are the result of impact (cars, etc.), insufficient concrete cover over the reinforcing steel or poor quality control of workmanship or materials. Standardized symbols for concrete spalls can be referenced in Deficiency and References Legends, Appendix H and in Table 16-1.

An example of typical structural defects documented using standard inspection parameters is shown below. In this example, a concrete spall located at a construction joint on the right wall panel at station 250+55 is 2-square feet in surface area, 4-inches deep, with exposed reinforcing steel (rebar) (R) which has a section loss of approximately 20% and has a glistening surface of water (GS) is documented as follows:

Station	**Location**	**Type**	**Area (depth)**	**Re-Rod**	**Moisture**	**Comments**
250+55	RW	S-2	2 S.F. (4”)	R-2, 20%	GS	At construction joint

Note: Typical Spall Classifications: S-1 Concrete spall less than 2”, S-2 Concrete spall to reinforcing steel ,S-3 Concrete spall behind reinforcing steel, S-4 Special concrete spall

Lists of standardized identification codes for deficiencies are included in Appendix H.

16.3 GROUNDWATER INTRUSION

16.3.1 General

Groundwater intrusion can be mitigated either by treating the ground outside the tunnel or by sealing the inside of the tunnel. This section will deal with the sealing of an existing lining rather than formation grouting outside of the tunnel.

The selection of the proper repair product for the conditions found on the project is key to the success of a leak containment program. Each site has its own particular environmental and physical properties. The pH, hardness, chemical composition, turbidity of the groundwater entering the tunnel all contribute to the ability of the chemical or particle grouts to effectively seal the leaking defect. The physical conditions that created the defect, movement of the crack or joint, the potential for freezing and the amount of water inflow all are site specific constraints for the selection of the repair material and all of these parameters must be assessed. Ideally, if any movement of the crack or joint is suspected it is best to monitor the defect for a period of time sufficient to allow for an estimation of actual movement.

The selection of the proper grout to seal a tunnel liner is dependent on the degree of leakage into the tunnel from the defect. Typically the tunnel defects that cause leakage are construction joints liner gaskets, and cracks that are the full depth of the liner. Standardized terms have been developed to describe the inflow of water. Standardized terms are useful in the selection of the grout because they allow all personnel including individuals who have not visited the tunnel to be familiar with the degree of water inflow. This familiarity of all personnel including the grout manufacturer facilitates the selection of the proper product and procedure for sealing the leak lists common terms used for the identification of leakage in the United States.

Table 16-1 Common U.S. Descriptions of Tunnel Leakage (Russell, 1992)

Item	Symbol	Description
Moist	M	Discoloration of the surface of the lining, moist to touch
Past Moisture	PM	Area showing indications of previous wetness, calcification etc
Glistening Surface	GS	Visible movement of a film of water across a surface
Flowing	F	Continuous flow of water from a defect; requires volume measurement
Dry	D	Structural defect illustrates no signs of moisture

16.3.2 Repair Materials

The selection of the proper repair product for the site-specific condition is key to the successful repair of a tunnel or underground structure leak. The most common way to seal a tunnel liner is to inject a chemical or cementitious grout. The grout can be applied to the outside of the tunnel to create a “blister” type repair that seals off the leak by covering the affected area with grout. The selection of the grout is dependent on the groundwater inflow and chemical properties from the soil and water.

The most common method of sealing cracks and joints that are leaking is to inject a chemical or particle grout directly into the crack or joint. This is accomplished by drilling holes at a 45 degree angle through the defect. The holes are spaced alternately on either side of the defect at a distance equal to ½ the thickness of the structural element. The drill holes intersect the defect and become the path for the injection of the grout into the defect. All holes must be flushed with water to clean any debris from the hole and to clean the sides of the crack or joint prior to injection to ensure proper bonding of the grout to the concrete. Typical injection ports are shown in Figure 16-3. Figure 16-4 shows field injection of grout. Figure 16-5 illustrates the typical location of injection ports and leaking crack repair detail (FHWA, 2005b).

Figure 16-3 Typical Injection Ports for Chemical Grout

Figure 16-4 Leak Injection, Tuscarora Tunnel PA Turnpike

The selection of the grout is dependent on the width, moisture content, and potential for movement within the crack or joint. For joints that move, only chemical grout is appropriate. The movement of the joint or crack will fracture any particle grout and will cause the leak to reappear. Single component water reactive polyurethane chemical grout is the most effective grout for the full depth sealing of cracks and joints that have moisture present within the defect. If the defect is subject to seasonal wetness and is dry at the time of repair a hydrophilic grout should be used. When utilizing a hydrophilic grout, water must be introduced into the defect to catalyze the grout. Hydrophobic grouts have a catalyzing agent injected with the chemical grout or premixed into the grout prior to injection. In both cases water or a catalyst is used to gel the grout. Alternatively, hydrophobic chemical grout may be utilized. Hydrophobic chemical grouts rely upon a chemical reaction to cure whereas hydrophilic chemical grout require water to catalyze. Common hydrophobic grouts are acrylates and closed cell polyurethane. The installation of both types of grout is similar to that described here.

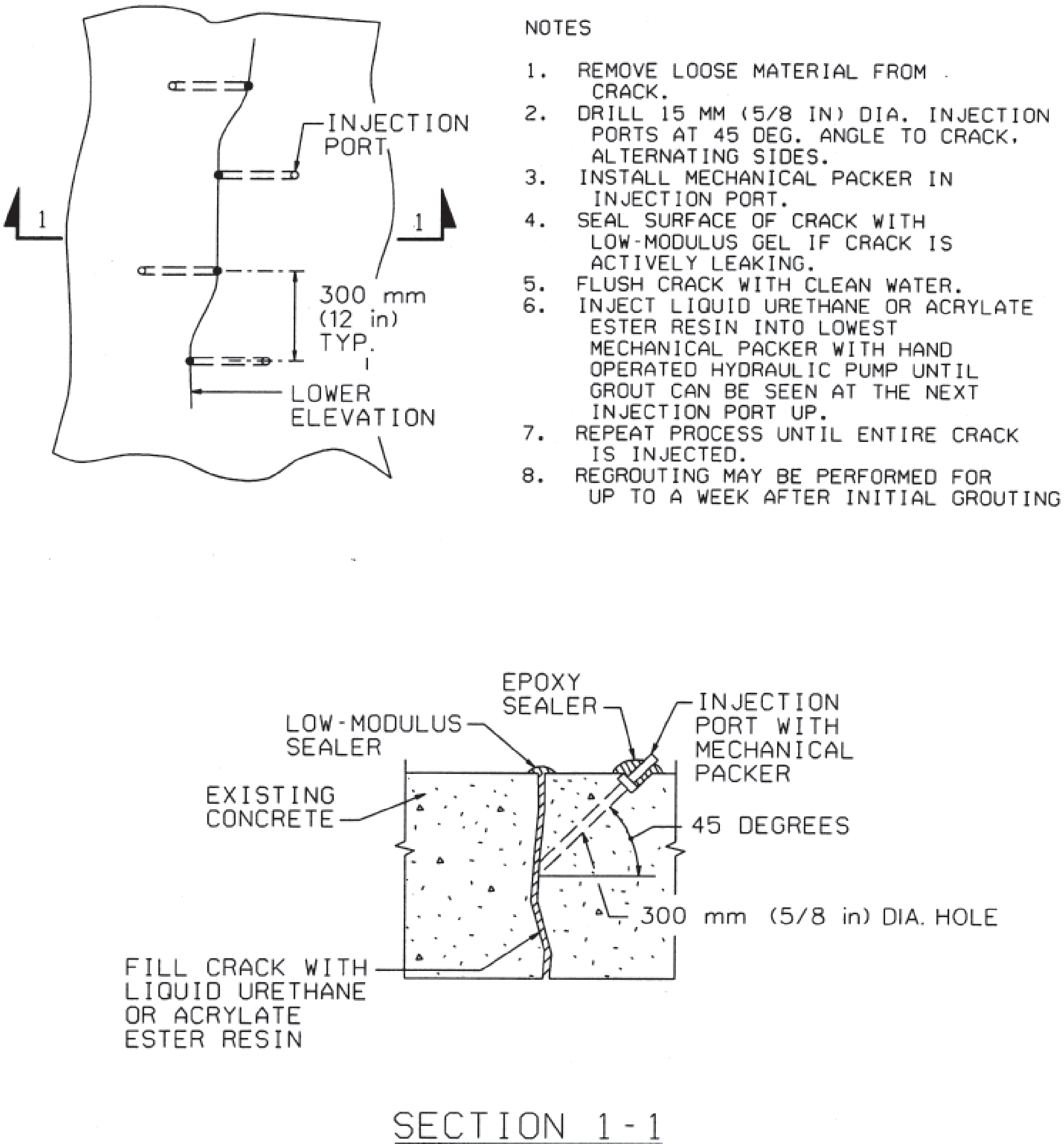


Figure 16-5 Typical Location of Injection Ports and Leaking Crack Repair Detail (FHWA, 2005b)

In situations where the defect is not subject to movement and is dry at the time of repair an epoxy grout can be injected into the defect in the same manner that concrete is structurally rebonded. The grouts shown in Table 16-2 are typical grouts for the injection cracks and joints in a tunnel liner. The particle grouts are often used for formation grouting outside of the tunnel liner or in very large dry cracks and joints. The most commonly used grouts for the sealing of cracks in tunnel liners are the polyurethanes and acrylates.

Table 16-2 Typical Grouts for Leak Sealing (Russell, 1992)

Description	Viscosity	Toxicity	Strength	Remarks
Particle Grout				
Flyash Type F;C	Med (50cps-2:1)	Low	High	Non flexible
Type I Cement	Med(50Cps-2-1)	Low	High	Non flexible
Type III Cement	Med (15cps-2:1)	Low	High	Non flexible
Microfine Cement	Low (8cps-2:1)	Low	High	Non flexible
Bentonite	Med (50 cps 2:1)	Low	Low	Semi flexible
Chemical Grout				
Acrylamides	Low (10 cps 2:1)	High	Low	Flexible
Acrylates	Low (10 cps)	Low	High	Semi flexible -No shrinkage: Good success record
Silicates	Low (6cps)	Low	High	Non flexible- High Shrinkage
Lignosulfates	Low (8 cps)	High	Low	Flexible not widely used
Polyurethane (MDI)	High (400 cps)	Med.	Low	Flexible: Good success record (Hydrophilic)
Polyurethane (TDI)	High (400 cps)	Med.	Low	Flexible : Good success Record(Hydrophobic)

Porous concrete can be sealed from the interior (negative side) of the tunnel to provide for a waterproof seal within the tunnel. Crystalline cementitious grouts that are applied to the interior of the tunnel and kept moist for 72 hours after application form a chemical bond with the free lime in the concrete and reduce the pore size of the concrete such that the free water vapor in the concrete cannot pass through. The success of these materials is varied and is to be used when no other alternative is available.

Interior side waterproofing is also performed by covering the interior surface of the wall with a cementitious coating consisting of two 1/8-inch thick coats applied to a moist concrete surface. Figure 16-6 illustrates the success of this type of coating in a tunnel in Pennsylvania with an external hydrostatic pressure of approximately 400 feet of water.

16.4 STRUCTURAL REPAIR – CONCRETE

16.4.1 Introduction

The repair of concrete delaminations and spalls in tunnels has traditionally been performed by the form-and-pour method for the placement of concrete, or by the hand application of cementitious mortars that have been modified by the addition of polymers. Both of these methods are not well suited for highway tunnels that are in continuous daily operation. This daily operation usually permits the tunnel to be out of service for very short periods of time. Therefore, the repair process must be rapid, not infringe on the operating envelope of the daily traffic and be a durable long-term monolithic repair.

Figure 16-6 Negative Side Cementitious Coating, Tuscarora Tunnel PA Turnpike

Today, the repair of concrete structural elements is performed typically by two methods: the use of hand applied mortars for small repairs and the use of shotcrete for larger structural repairs. In either case the preparation of the substrate is the same, only the type of material differs.

Shotcrete (also discussed in Chapters 9 and 10), is the pneumatic application of cementitious products which can be applied to restore concrete structures. This process has been in use for over decades in the US for the construction and repair of concrete structures both above and below ground. Shotcrete is defined by the American Concrete Institute as a "Mortar or concrete pneumatically projected at a high velocity onto a surface." Since the 1970's the use of low-pressure application of cementitious mortar has been commonplace in Europe and is known as Plastering. Over the years, developments in materials and methods of application have made the use of polymer cementitious shotcrete products for the repair of defects in tunnel liners in active highway tunnels cost effective. The selection of the process type, and the material to be applied is dependent on the specific conditions for tunnel access and available time for the installation of the repair. Shotcrete is preferred to other repair methods since the repair is monolithic and becomes part of the structure. The use of shotcrete is a process that allows for rapid setup, application and ease of transport into and out of the tunnel on a daily basis.

This section only provides the procedures utilized to delineate the extent of the repairs to the liner, and the work required to implement the shotcrete repairs. Refer to Chapter 10 for a more general discussion regarding shotcrete. Table 16-3 lists the most commonly used materials for the repair of tunnel liners.

Table 16-3 Comparison of Repair Materials (Russell, 2007)

Application	Two-Component Self Leveling Mortar	Polymer Shotcrete Wet Process	Two Component Mortar	Polymer Shotcrete Dry Process	Polymer Masonry Mortar
On Grade; above, below	yes	yes	yes	yes	yes
On horizontal	yes	yes	yes	yes	yes
On vertical	no	yes	yes	yes	yes
Overlay system	yes	No	yes	no	yes
Structural repair	yes	yes	yes	yes	yes
Leveling material	yes	yes	no	yes	yes
Filler: voids	no	yes	yes	yes	yes
Maximum depth	3 inches	unlimited	1 inch/lift	unlimited	1 inch/lift
Minimum depth	½ inch	¼ inch	1/4 inch	¼ inch	1/8 inch
Extended w/ aggregate	yes	No	yes	no	yes
High abrasion	yes	yes	yes	yes	yes
Good bond Strength	yes	yes	yes	yes	yes
Compatible coefficient of expansion w/concrete	yes	yes	yes	yes	yes
Resistant to salts	yes	yes	yes	yes	yes
High early strength	yes	yes	yes	yes	yes
High Flexural	yes	yes	Yes	yes	yes
Good freeze- thaw	yes	yes	yes	yes	yes
Vapor Barrier	yes	No	no	no	no
Flammable	no	No	no	no	no
Ok Potable water	yes	yes	yes	yes	yes
Open to traffic 1-2 hours	yes	yes	yes	yes	yes
Low rebound dust	yes	yes	yes	no	yes
Prepackaged	yes	yes	yes	yes	yes

16.4.2 Surface Preparation

The surface preparation for concrete repair requires removal of all unsound concrete by either the use of chipping hammers or the use of hydro-demolition. Unsound concrete is removed to the full depth of the unsound concrete. In cases where chipping hammers are used it has been found that limiting the size of the hammers by weight is the best way to control over excavation. Limiting the weight of the chipping hammers with bit, to less than 30 lbs. (13.6Kg) reduces the risk of over excavation of concrete. These hammers are too weak to excavate concrete in excess of 4,000 psi. (27,580 Kpa). The use of Hydro-demolition requires testing on site, at the beginning of the project to determine what pressures are required to excavate the unsound concrete without removing the sound substrate (Figure 16-7).

Hydro-demolition should not be used in areas that house electrical equipment, cables, or other mechanical equipment that may be effected by the excavation process. The area to be repaired must not have feather edges, and must have a vertical edge of at least 1/8 inch in height. This vertical shoulder is necessary to prevent spalling at the edge of the new repair.

Figure 16-7 Substrate after Hydro-demolition, Shawmut Jct. Boston

After the unsound concrete is removed, any leaking cracks or construction joints must be sealed prior to the application of the reinforcing steel coatings and the shotcrete. This sealing should be performed using a chemical grout suitable for the type and magnitude of the leakage. In general single component polyurethane grouts are the most successful in effectively sealing most tunnel leaks. Refer to Section 16.3.2 for more information on sealing leaks.

16.4.3 Reinforcing Steel

Once the unsound concrete has been removed, reinforcing steel must be cleaned and if a loss of section is evident the damaged reinforcing steel must be removed and replaced. All rust and scale must be removed from the reinforcing steel and any exposed steel liner sections or other structural steel elements. The cleaning is generally to a white metal commercial grade cleaning. Once cleaned the reinforcing steel is to be evaluated for loss of section and if the loss of section is greater than 30% an analysis must be performed. If the results of the analysis indicate that the lining does not have adequate strength with the remaining reinforcing steel, then the damaged steel must be replaced. Mechanical couplers are used when splicing new reinforcing steel to existing. Mechanical couplers eliminate the need for lap splices in the reinforcing steel and thereby reduce the amount of lining removal required to replace the reinforcing steel. (Figure 16-8)

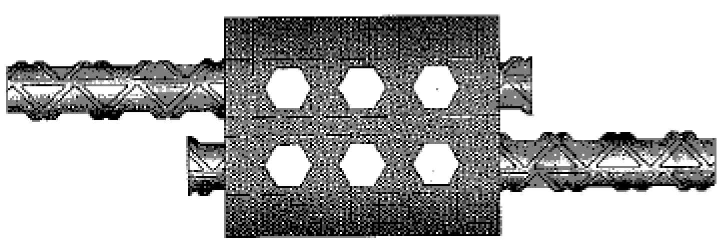


Figure 16-8 Typical Mechanical Coupler for Reinforcing Steel

After the steel has been cleaned a coating must be placed on the steel to protect the steel from accelerated corrosion due to the formation of an electrolytic cell. Numerous products exist for this purpose, including epoxy and zinc rich coatings. Zinc rich coatings are better suited for this application due to the fact that they do not form a bond-breaker as do many epoxies. This is important since these materials are applied by the use of a paint brush and it is difficult to prevent the concrete surface from being accidentally coated. The application of the zinc rich coating is to be performed within 48 hours of the cleaning and not more than 30 days prior the application of the shotcrete.

16.4.4 Repairs

Small shallow spalls are repaired by the use of a polymer modified hand patch mortar as shown in Figure 16-9. Hand patch mortar is a prepackaged polymer modified mortar that is applied in lifts of 1 to 2 inches. The patch areas are generally less than 2 square feet in area and require keying into the substrate by the use of "j" hooks and welded wire mesh or rebar. Unsound concrete is removed by either a hydro-demolition hand wand or by a chipping hammer with a weight of less than 30 lbs, including bit. The limiting of the hammer size provides for the removal of concrete of less than 4,000 psi compressive strength and limits over excavation since the hammer energy is not sufficiently strong to remove higher strength concrete.

Other than small repairs which utilize the repair mortars, the most commonly used material is shotcrete (or specifically prepackaged polymer modified fibrous shotcrete). Figure 16-10 illustrates the details of typical concrete repairs for deeper spalls. Discussions of the deeper spall repairs are included in Section 16.4.5 Shotcrete Repair.

16.4.5 Shotcrete Repairs

As discussed in Chapter 10, there are two processes for the application of shotcrete; Dry Process and Wet Process. Both processes have been in use for many years and are equally applicable for use in tunnel rehabilitations. The wet process creates little dust and is applicable for use in tunnels when partial tunnel closures allow traffic inside the tunnel during the repair work. The dry process creates extensive dust and is not suitable for partial tunnel closures due to limited visibility created by the dust.

The successful application of shotcrete regardless of the process chosen relies on the skill of the nozzleman (Figure 16-11) (In the case of the wet process both the nozzleman and the laborer mixing the mortar). A successful repair program requires the nozzleman and the other members of the shotcrete crew to be skilled and tested on site using mock-ups of the types of areas to be repaired. These mock-ups should closely duplicate the shape and surfaces to be repaired. This testing program is often used to certify the skill of the shotcreting crew and provides for better quality control during the progress of the work. The testing program develops an understanding between the Engineer, Owner and Contractor that defines an acceptable product for the work.

Once the reinforcing and structural steel elements have been cleaned and coated, welded wire mesh is to be placed over the area to be shotcreted (Figure 16-12). The mesh is placed to within 2 inches of the edge of the repair. The wire mesh is attached to the existing reinforcing and to the substrate by the use of "J" hooks.

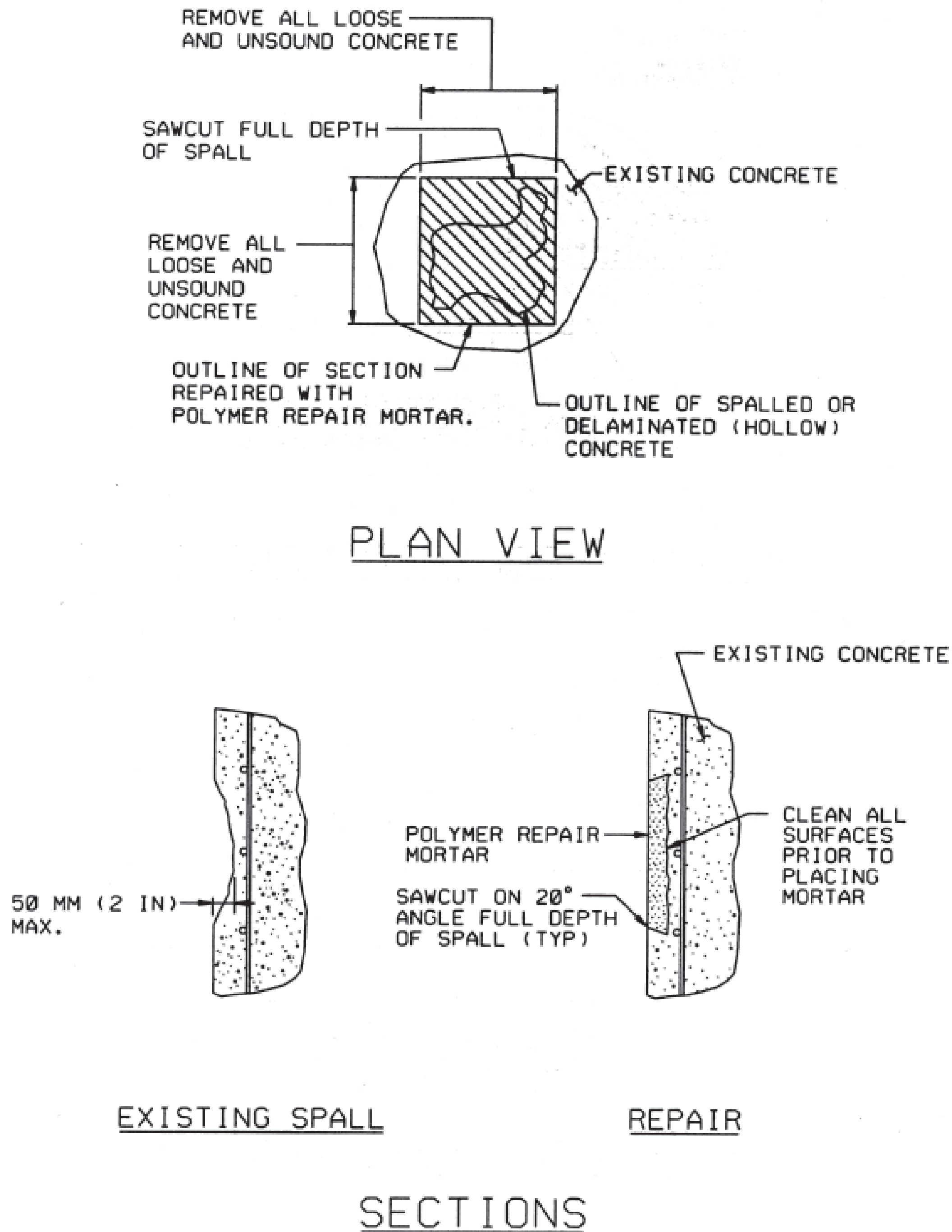


Figure 16-9 Shallow Spall Repair (FHWA, 2005b)

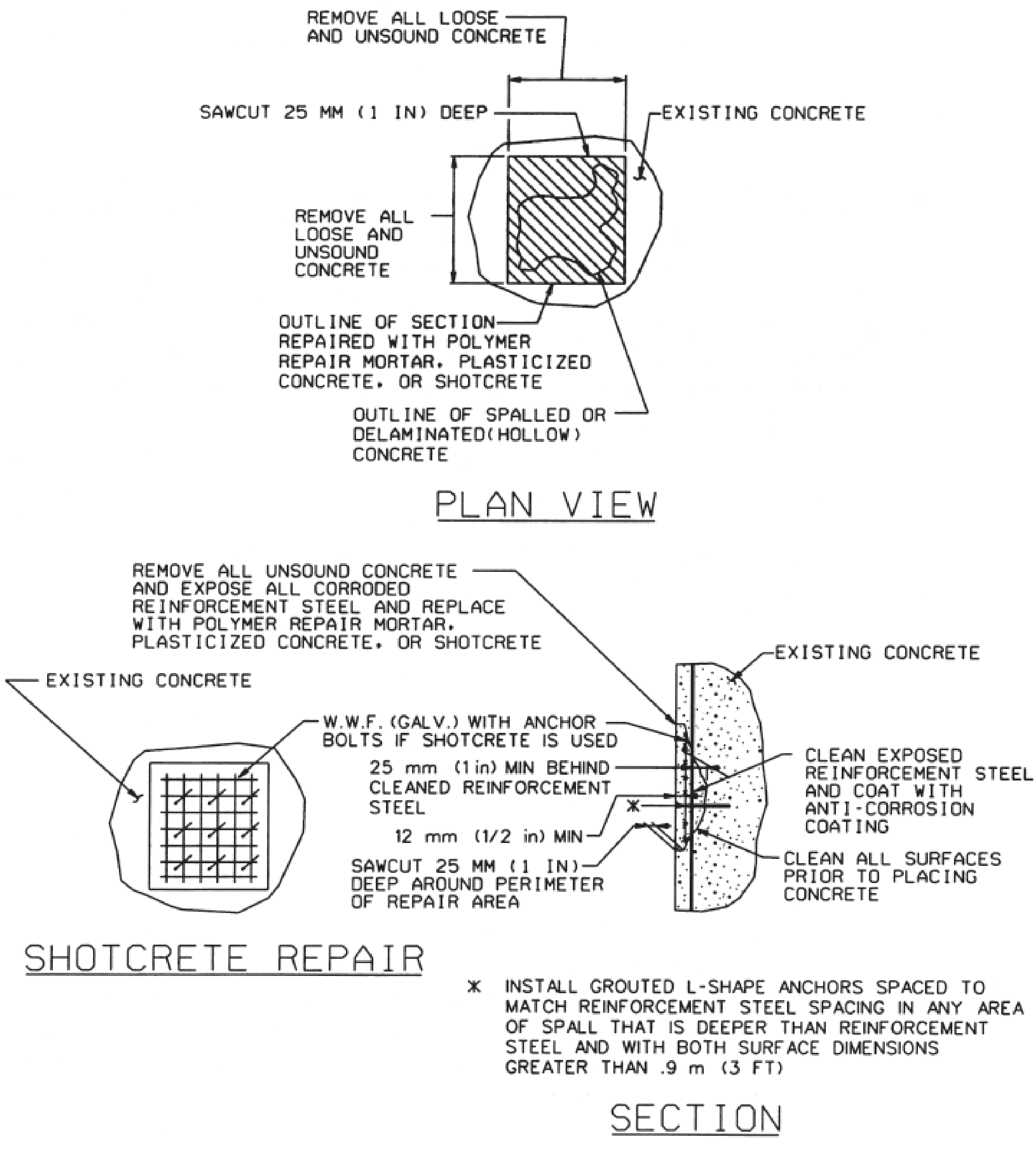


Figure 16-10 Typical Sections at Concrete Repair (FHWA, 2005b)

The purpose of the wire mesh is to assist in the buildup of the shotcrete and to provide for a monolithic repair that becomes part of the host structure. The wire mesh should be hot dipped galvanized after fabrication and is best if delivered to the site in sheets rather than on a roll. If epoxy coated mesh is used it must be in sheets in order to eliminate field touch-up of the cut ends of the mesh. The mesh size for dry process is a 2 X 2 inch mesh and for wet process 4 X 4 inch mesh. The larger mesh is required for the wet process to prevent clogging of the mesh by the shotcrete and therefore creating voids behind the mesh surface.

Figure 16-11 Nozzleman Applying Wet Process Shotcrete, USPS Tunnel Chicago

Figure 16-12 Reinforcing Steel for Repair, Sumner Tunnel Boston

After the entire area to be patched is filled with shotcrete the material is allowed to cure for 20-30 minutes, at which time the mix is screeded and troweled to the desired finish (Figure 16-13). Trying to work the shotcrete prior to this time will result in tearing of the surface and make finishing very difficult. Caution must be exercised to monitor the drying rate of the shotcrete since the times stated here will vary depending on wind conditions and relative humidity. After the repair has been troweled to the desired finish a curing compound must be sprayed on the surface of the new shotcrete to prevent rapid drying. The manufacturer of the premixed shotcrete will recommend a curing compound best suited for the job site conditions.

Figure 16-13 Shotcrete Finishing, Shawmut Jct. Boston

16.5 STRUCTURAL INJECTION OF CRACKS

Cracking is the most common defect found in concrete tunnel liners. While most of the cracks are a result of thermal activity, there are cracks that are a result of structural stresses that were not accounted for in the design. It is important to note that cracks also occur as a result of shrinkage and thermal stresses in the tunnel structure. Cracks that exhibit thermal stresses should not be structurally rebonded since they will only move and re-crack. However, structural cracks that occur as a result of structural movement, such as settlement and are no longer moving should be structurally rebonded. Any crack being considered for structural rebonding must be monitored to assess if any movement is occurring. A structural analysis of the tunnel lining should be performed to ascertain if the subject crack requires rebonding.

There are three types of resin typically available for injection of structural cracks in tunnels. They are:

- Vinyl Ester Resin
- Amine Resin
- Polyester Resin

Vinyl ester resin is the common type of resin used for bridge repair work and is usually not suited for tunnel work since most cracks in tunnels are damp or wet. The vinyl ester resin will not bond to surface saturated concrete and will not structurally rebond a damp or moist crack. However, if the crack is totally dry during the injection process this epoxy will provide a suitable rebonding of the concrete.

Amine and polyester resins are best suited for the structural rebonding of cracks in tunnels. Both resins are unaffected by moisture during installation and will bond surface saturated concrete. Cracks with flowing water must be carefully injected and the manufacturer’s advise must be obtained to ensure proper installation of the resin.

In all cases the manufacturer’s recommendations must be followed for the injection of epoxy resins, particularly in the case of overhead installation. Figure 16-14 illustrates a typical installation of epoxy resin for the structural rebonding of cracks in concrete. The procedure for rebonding masonry and precast concrete elements is similar.

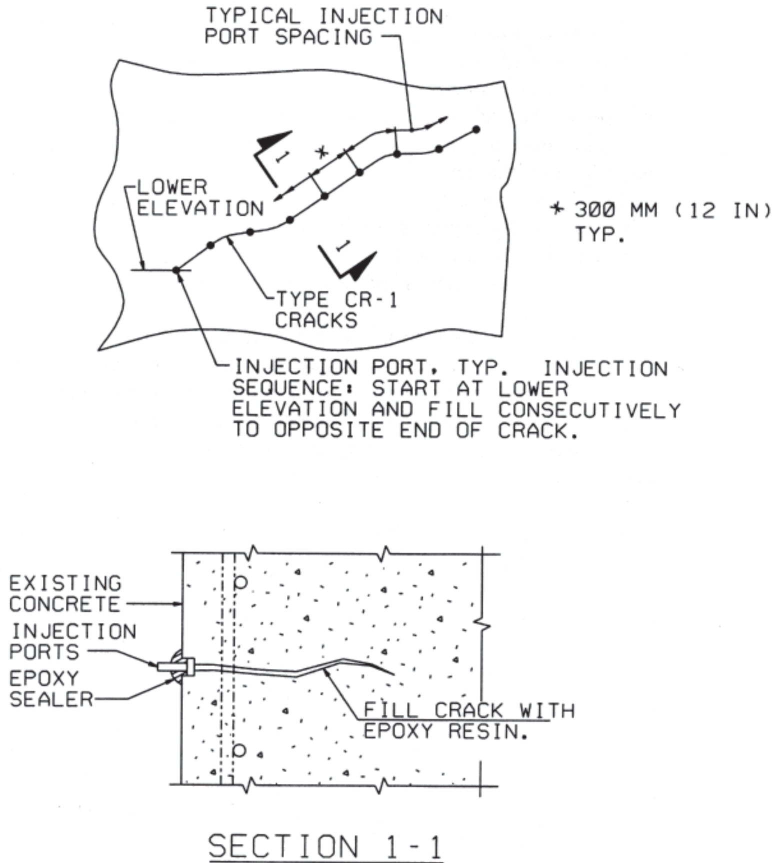


Figure 16-14 Typical Structural Crack Injection (FHWA, 2005b)

16.6 SEGMENTAL LININGS REPAIR

As discussed in Chapter 10, segmental lining can be made up of either, precast concrete, steel or cast iron. A segmental liner is usually the primary liner of a tunnel. The segments are either bolted together or keyed. The only segmental liners that are keyed are the precast liners. The most common problems with segmental liners is deformation of the flanges in the case of steel and cast iron liners and corner spalling of precast concrete segments. The spalling of precast segments and deformation of the flanges of steel/cast iron segments usually occurs at installation or as a result of impact damage from vehicles. In addition the rusting through of the liner plate of steel/cast iron segments occasionally occurs.

16.6.1 Precast Concrete Segmental Liner

The repair of spalls in precast concrete liner segments is performed by the use of a high performance polymer modified repair mortar which is formed to recreate the original lines of the segment. In the event the segment gasket is damaged the gasket's waterproofing function is restored by the injection of a polyurethane chemical grout as described above. Damaged bolt connections in precast concrete liner segments are repaired by carefully removing the bolt and installing a new bolt, washer, waterproof gasket and nut. The bolts are to be torqued to the original specification and checked with a torque wrench.

16.6.2 Steel/Cast Iron Liner

The repair of steel/cast iron liners varies according to the type of liner material. Steel, if made after 1923, is weldable while cast iron is not. Common defects in these types of liners are deformed flanges and penetration of the liner segment due to rusting. Deformed flanges can be repaired by reshaping the flanges with hammers or heat. Holes in steel liner segments can be repaired by welding on a new plate. Bolted connections often have galvanic corrosion which is caused by dissimilar metal contact and often require the entire bolted connection to be replaced. When the bolted connection is replaced a nylon isolation gasket is used to prevent contact between the high strength bolt and the liner plate. Figure 16-15 shows the repair of a rusted through steel segment and a repaired bolted connection.

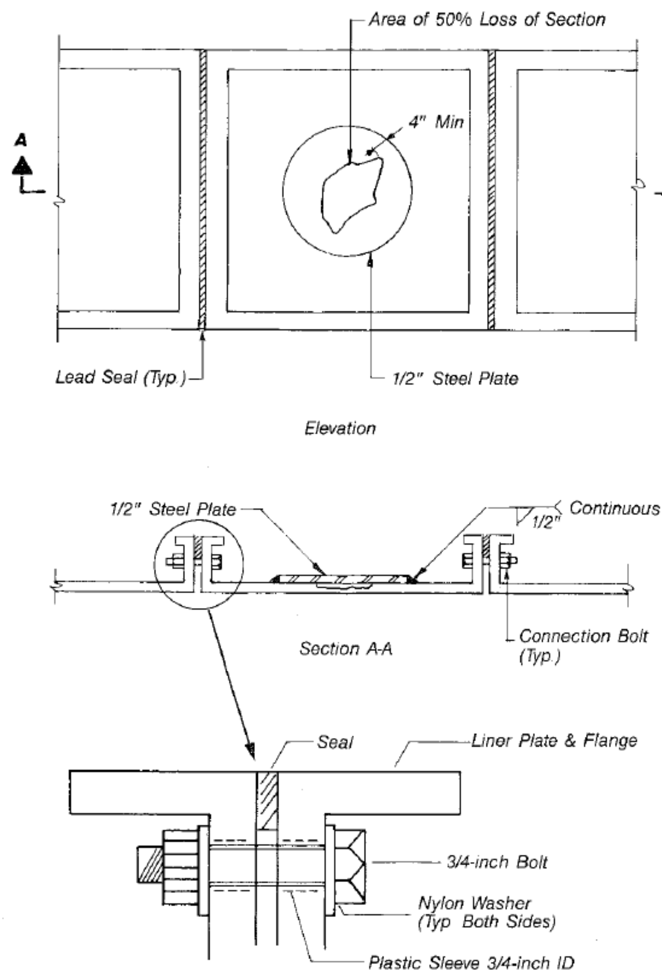


Figure 16-15 Steel Segmental Liner Repair (Russell, 2000)

Repairs to cast iron liner segments is similar to those for steel. However, since cast iron cannot be welded the repair plate for the segment is installed by brazing the repair plate to the cast iron or drilling and tapping the liner segment and bolting the repair plate to the original liner segment. In some instances it is easier to fill the area between the flanges with shotcrete. Figure 16-16 illustrates a test panel for filling a liner plate with shotcrete.

Figure 16-16 Cast Iron Segmental Segment Mock-up of Filling with Shotcrete, MBTA Boston

16.7 STEEL REPAIRS

16.7.1 GENERAL

Structural steel is commonly used at the portals of tunnels, support of internal ceilings, columns, segmental liners and as standoffs for tunnel finishes. The repairs to steel elements is to be site specific and to be performed in accordance with the appropriate standard (Figure 16-7). The American Welding Society's Standard Structural Steel Welding Code *AWS D1.1/D1.1 Structural Welding Guide* most recent version should be utilized for the construction of all welded steel connections. Repairs to Rivets and bolting must comply with AASHTO Specification.

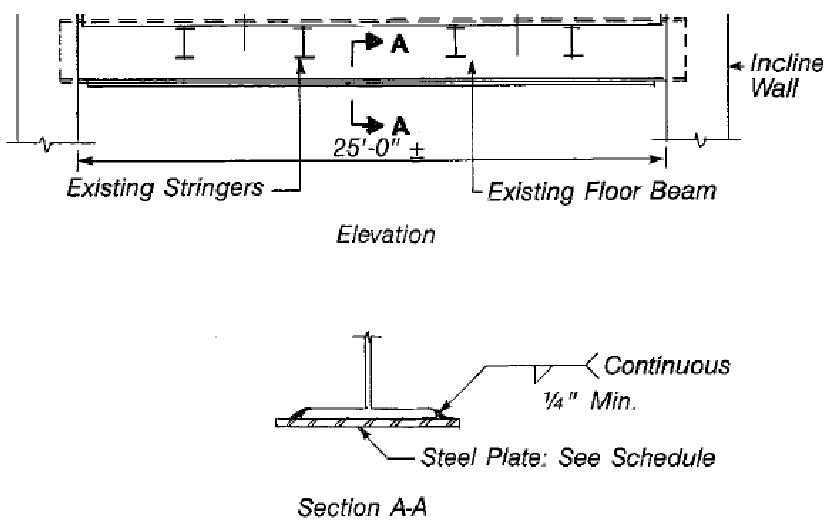


Figure 16-17 Typical Framing Steel Repair at Temporary Incline

16.8 MASONARY REPAIR

The restoration of masonry linings composed of clay brick or Ashlar (dimension) stone consists of the repointing of deficient mortar. As shown in Figure 16-18, the repointing of masonry joints consists of raking out the joint to a depth of approximately one inch (2.54cm). Once the joint has been raked clean and all old mortar removed, the joints are repointed with a cemtitious mortar, or a cementitious mortar that has been fortified with an acrylic bonding agent.

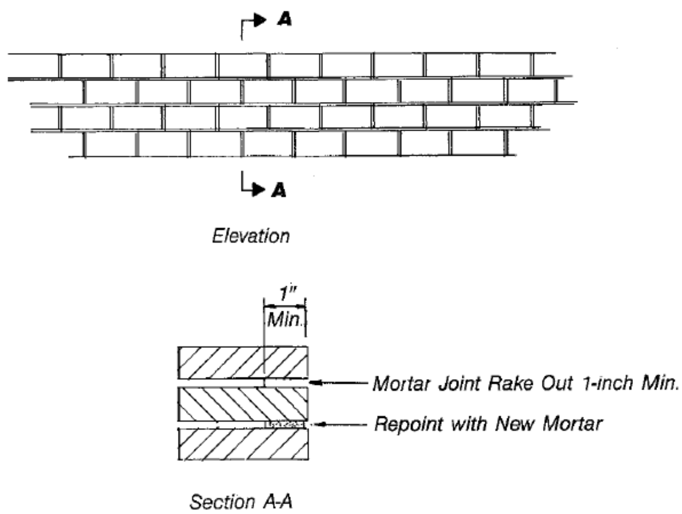


Figure 16-18 Typical Masonry Repair

Replacement of broken, slaked or crushed clay brick requires a detailed analysis to determine the causes and extent of the problem. Once the problem is properly identified a repair technique can be designed for the particular structure. Caution must be exercised in the removal of broken or damaged brick. The removal of numerous bricks from any one section may cause the wall or arch to fail. Therefore it is imperative that any repair work on masonry be performed by competent personnel having experience in the restoration of brick and stone masonry.

16.9 UNLINED ROCK TUNNELS

Unlined rock lined tunnels do not required a permanent concrete, brick or steel lining since the rock was competent and illustrated sufficient strength with minimal reinforcement to remain standing. These roadway tunnels are also usually very short in length. Most have support consisting of various types of rock reinforcement; including rock dowels, rock bolts, cable bolts and other reinforcement which were placed at various angles to cross discontinuities in the rock mass. These rock reinforcement elements typically range in length for 5 to 20 feet in length and are installed and grouted with resin or cementitious grout. Please refer to Chapter 6 for more detailed discussions about various types of rock reinforcement elements.

Rock reinforcement elements, may deteriorate and loose strength due to the corrosive environment and exposure typical in tunnels, and require replacement and installation of new rock reinforcement elements. Replacement of rock reinforcement elements requires a detailed investigation of the structural geology of the tunnel which is performed by an engineering geologist or geotechnical engineer having experience in geologic mapping and the rock stability analysis as discussed in Chapter 6.

Another more frequent cause for the need to repair unlined rock tunnels is the falling of rock fragments which over time become loose and drop onto the roadway. There are many ways to prevent this from occurring, the most common of which is to scale (remove) all loose rock on a periodic basis from the tunnel roof and walls by the use of a backhoe or hoe ram. Other methods include the placement of steel liner roof as a shelter, additional rock bolts and wire mesh to contain the falling rock fragments, and shotcrete on the areas of concern as shown in Figure 16-19 and Figure 16-20.

Figure 16-19 Rock Tunnel with Shotcrete Wall Repair and Arch Liner (I-75 Lima Ohio)

Figure 16-20 Rock Bolts (Dowels) Supporting Liner, I-75 Lima Ohio Underpass

16.10 SPECIAL CONSIDERATIONS FOR SUPPORTED CEILINGS/ HANGERS

Numerous highway tunnels in the United States have suspended ceilings for ventilation purposes and in some cases aesthetics. These ceilings are generally supported by keyways in the tunnel walls and by hanger rods that are attached to the tunnel liner either by means of cast-in-place inserts or post-installed mechanical or adhesive (chemical) anchors. FHWA issued a Technical Advisory in 2008 strongly discouraging the use of adhesive anchors for permanent sustained tension or overhead applications (see Appendix I). Any use of adhesive anchors in road tunnels must conform to current FHWA directives and other applicable codes and regulations.

The inspection of these hangers is important to tunnel safety and a rigorous and regular inspection program that considers importance and redundancy is strongly recommended to maintain an appropriate level of confidence in their long-term performance.

During inspection one method used to verify hangers are in tension is by "ringing" each hanger. Ringing a hanger is done by striking it with a masons hammer. A hanger in tension will vibrate or ring like a bell after being struck while a hanger that is not in tension because of a connection or other defect will not ring. Hangers that exhibit a defect or lack of tension should be closely inspected and checked for structural suitability. Examples of typical hangers and their components are shown in Figure 16-21.

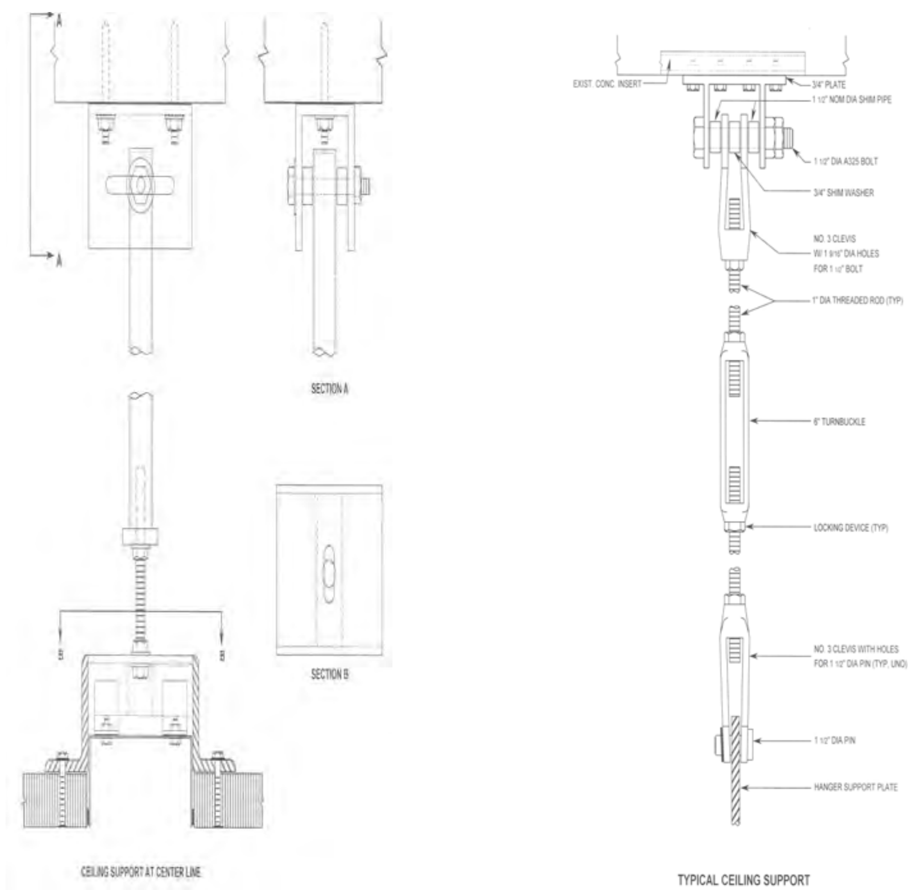


Figure 16-21 Typical Hanger Supports for Suspended Ceiling

The repair of ceiling hangers depends on the particular type of defect. If the hanger rod, clevis, turnbuckle or connection pins are broken or damaged they can be simply replaced with similar components which are readily available from many sources, including most large hardware supply retailers (Figure 16-22).

Figure 16-22 Typical Replacement Hanger Hardware

The repair of loose connections at the tunnel arch is of primary concern. The recommended repair for failed adhesive anchors is to replace them with undercut mechanical anchors typical examples of which are shown in Figure 16-23.

Figure 16-23 Typical Undercut Mechanical Anchors

GLOSSARY

AASHTO	American Association of State Highway and Transportation Officials
ANFO	Ammonium nitrate mixed with fuel oil used as an explosive in rock excavation.
ASCE	American Society of Civil Engineer
Accelerator	Admixture to accelerate the process of hardening.
Admixtures	Materials in liquid or powder form, added to the shotcrete mix influencing the chemical process and consistency of sprayed concrete.
Aggregates	Graded mixture of mineral components being added to a concrete mix.
Alluvium	A general term for recent deposits resulting from streams.
Aquiclude	1. Rock formation that, although porous and capable of absorbing water slowly, does not transmit water fast enough to furnish an appreciable supply for a well or spring. 2. An impermeable rock formation that may contain water but is incapable of transmitting significant water quantities. Usually functions as an upper or lower boundary of an aquifer.
Aquifer	1. A water-bearing layer of permeable rock or soil. 2. A formation, a group of formations, or a part of a formation that contains sufficient saturated permeable material to yield significant quantities of water to wells and springs.
Aquitard	A formation that retards but does not prevent water moving to or from an adjacent aquifer. It does not yield water readily to wells or springs, but may store groundwater.
Artesian condition	Groundwater confined under hydrostatic pressure. The water level in an artesian well stands above the top of the artesian water body it taps. If the water level in an artesian well stands above the land surface, the well is a flowing artesian well.
Bench	A berm or block of rock within the final outline of a tunnel that is left after a top heading has been excavated.
Bit	A star or chisel-pointed tip forged or screwed (detachable) to the end of a drill steel.
Blocking	Wood or metal blocks placed between the excavated surface of a tunnel and the bracing system, e.g., steel sets. Continuous blocking can also be provided by shotcrete.

Bootleg or Socket	That portion or remainder of a shot hole found in a face after a blast has been fired.
Breast boarding	Partial or complete braced supports across the tunnel face that hold soft ground during tunnel driving.
Bulkhead	A partition built in an underground structure or structural lining to prevent the passage of air, water, or mud.
Burn cut	Cut holes for tunnel blasting that are heavily charged, close together, and parallel. About four cut holes are used that produce a central, cylindrical hole of completely shattered rock. The central or bum cut provides a free face for breaking rock with succeeding blasts.
CCTV	Closed-circuit television
CFD	Computational fluid dynamics
Chemical grout	A combination of chemicals that gel into a semisolid after they are injected into the ground to solidify water-bearing soil and rocks.
Cherry picker	A gantry crane used in large tunnels to pick up muck cars and shift a filled car from a position next to the working face over other cars to the rear of the train.
Cohesion	A measure of the shear strength of a material along a surface with no perpendicular stress applied to that surface.
Conglomerate	A sedimentary rock mass made up of rounded to subangular coarse fragments in a matrix of finer grained material.
Controlled blasting	Use of patterned drilling and optimum amounts of explosives and detonating devices to control blasting damage.
Cover	Perpendicular distance to nearest ground surface from the tunnel.
Crown	The highest part of a tunnel.
Cut-and-cover	A sequence of construction in which a trench is excavated, the tunnel or conduit section is constructed, and then covered with backfill.
Cutterhead	The front end of a mechanical excavator, usually a wheel on a tunnel boring machine, that cuts through rock or soft ground.
Delays	Detonators that explode at a suitable fraction of a second after passage of the fling current from the exploder. Delays are used to ensure that each charge will fire into a cavity created by earlier shots in the round.
Disk cutter	A disc-shaped cutter mounted on a cutterhead.
Drag bit	A spade-shaped cutter mounted on a cutterhead.

Drift	An approximately horizontal passageway or portion of a tunnel. In the latter sense, depending on its location in the final tunnel cross section, it may be classified as a "crown drift," "side drift" "bottom drift", etc. A small tunnel driven ahead of the main tunnel.
Drill-and-blast	A method of mining in which small-diameter holes are drilled into the rock and then loaded with explosives. The blast from the explosives fragments and breaks the rock from the face so that the reek can be removed. The underground opening is advanced by repeated drilling and blasting.
Dry Mix	Mixture being supplied to the nozzle where the required amount of water and, if required, liquid accelerator is added.
FHWA	Federal Highway Adminstration
Face	The advance end or wall of a tunnel, drift, or other excavation at which work is progressing.
Face stabilization wedge	Unexcavated portion of the heading temporarily left in place to enhance face stability.
Fibers	Steel fibers or synthetic fibers added to mixes to improve flexural strength and post failure characteristics of the shotcrete or concrete.
Final Lining	Cast-in-place concrete, shotcrete, precast concrete segment, or steel lining placed after installation of the initial support and waterproofing (if applicable).
Fiber Reinforced Shotcrete (FRS)	Shotcrete reinforced with either steel (SFRS) or synthetic fibers.
Finishing Shotcrete	Unreinforced sprayed concrete to smoothen rough or undulating surfaces or to cover steel fiber reinforced shotcrete. Typically applied on initial shotcrete lining in preparation for the waterproofing installation or as finishing layer for the final surface of permanent support linings.
Finite difference method	
Finite element method	The representation of a structure as a finite number of two-dimensional and/or three-dimensional components called finite elements.
Firm ground	Stiff sediments or soft sedimentary rock in which the tunnel heading can be advanced without any, or with only minimal, roof support, the permanent lining can be constructed before the ground begins to move or ravel.

Flashcrete (Sealing Shotcrete)	Typically unreinforced or steel fiber reinforced sprayed concrete layer to seal off exposed ground surface, typically 30 to 50 mm (1.2 to 2 in) thick.
Forepole	A pointed board or steel rod driven ahead of timber or steel sets for temporary excavation support.
Forepoling	Driving forepoles ahead of the excavation, usually supported on the last steel set or lattice girder erected, and in an array that furnishes temporary overhead protection while installing the next set.
Full-face Heading	Excavation of the whole tunnel face in one operation.
Gouge zone	A layer of fine, wet, clayey material occurring near, in, or at either side of a fault or fault zone.
Grade	Vertical alignment of the underground opening or slope of the vertical alignment.
Ground control	Any technique used to stabilize a disturbed or unstable rock mass.
Ground stabilization	Combined application of ground reinforcement and ground support to prevent failure of the rock mass.
Ground support	Installation of any type of engineering structure around or inside the excavation, such as steel sets, wooden cribs, timbers, concrete blocks, or lining, which will increase its stability. This type of support is external to the rock/soil mass.
Ground Support Class	Prescribed excavation sequence, support and local support based on the type of host material expected in excavation cross section as well as by the anticipated response and behavior of the host material during excavation.
Ground Support System	System of interacting support elements such as shotcrete lining, steel support, rock reinforcement (dowels, bolts, spiling, etc.) in combination with an excavation and support sequence. If required, ground support systems can be supplemented by ground improvement measures (e.g. grouting, ground freezing, dewatering).
Grout	Neat cement slurry or a mix of equal volumes of cement and sand that is poured into joints in masonry or injected into rocks. Also used to designate the process of injecting joint-filling material into rocks. See grouting.
Grouted Pipe Spiling	Perforated steel pipes installed at the tunnel heading ahead of excavation and grouted as a means to pre-support the ground.
Heading and bench	A method of tunneling in which a top heading is excavated first, followed by excavation of the horizontal bench.

Ho-ram	A hydraulically operated hammer, typically attached to an articulating boom, used to break hard rock or concrete.
Hydraulic jacking	Phenomenon that develops when hydraulic pressure within a jacking surface, such as a joint or bedding plane, exceeds the total normal stress acting across the jacking surface. This results in an increase of the aperture of the jacking surface and consequent increased leakage rates, and spreading of the hydraulic pressures. Sometimes referred to as hydraulic fracturing.
ITA	International Tunnel Association
Initial Shotcrete Lining	Shotcrete layer of a minimum thickness as defined in the ground support class typically reinforced with lattice girders, splice bars and either fibers (steel or synthetic) or welded wire fabric.
Initial Support	Support required to provide stability of the tunnel opening and to maintain the inherent strength of the ground surrounding the tunnel openings while preventing unnecessary loosening and enhancing the stress redistribution process. This function of support may be enhanced by installation of systematic Tunnel Pre-support and local support where required by ground conditions. It typically consists of reinforced shotcrete, rock reinforcement, pre-support, steel rib or lattice girder sets, or combinations thereof.
Invert	On a circular tunnel, the invert is approximately the bottom 90 deg of the arc of the tunnel; on a square-bottom tunnel, it is the bottom of the tunnel.
Invert strut	The member of a set that is located in the invert.
Joint	A fracture in a rock along which no discernible movement has occurred.
Jumbo	A movable machine containing working platforms and drills, used for drilling and loading blast holes, scaling the face, or performing other work related to excavation.
Jump set	Steel set or timber support installed between overstressed sets.
Lagging	Wood planking, steel channels, or other structural materials spanning the area between sets.
Length of Round	Length of the unsupported span of exposed ground opened up during one round of excavation, followed by the installation of the initial support to advance the tunnel.
Local Support	Non-systematic application of initial support measures in addition to the standard support and systematic pre-support as specified by the ground support class for local stabilization and safety during tunneling. Also referred to as additional initial support.

Liner Plates	Pressed steel plates installed between the webs of the ribs to make a tight lagging, or bolted together outside the ribs to make a continuous skin.
Lithology	The character of a rock described in terms of its structure, color, mineral composition, grain size, and arrangement of its component parts.
Mine straps	Steel bands on the order of 12 in. wide and several feet long designed to span between rock bolts and provide additional rock mass support.
Mining	The process of digging below the surface of the ground to extract ore or to produce a passageway such as a tunnel.
Mix	Mixture of cement, aggregates and, if required, chemical admixtures being processed in a batching plant.
Mixed face	The situation when the tunnel passes through two (or more) materials of markedly different characteristics and both are exposed simultaneously at the face (e.g., rock and soil, or clay and sand).
Mohr's hardness scale	A scale of mineral hardness, ranging from 1 (softest) to 10 (hardest).
Muck	Broken rock or earth excavated from a tunnel or shaft.
NCHRP	National Cooperative Highway Research Program
NFPA	National Fire Protection Association
NHI	National Highway Institute
Nozzle	Specially manufactured hose through which sprayed concrete is applied. Designed to add water (plus accelerator) through jets to the dry mix or add other admixtures to the wet mix.
Nozzleman	Person who applies the shotcrete by operating the nozzle.
Open cut	Any excavation made from the ground surface downward.
Overbreak	The quantity of rock that is actually excavated beyond the perimeter established as the desired tunnel outline.
Overburden	The mantle of earth overlying a designated unit; in this report, refers to soil load overlying the tunnel.
PIARC	World Road Association (previously the Permanent International Association of Roadways Congress)

Partial Drifts	To achieve an early, temporary ring closure and to reduce excavation face size, partial drifts such as sidewall drifts, middle drifts and top heading, bench, and invert drifts can be used. These partial drifts are supported by temporary shotcrete support, such as temporary middle walls, invert supports, etc.
Pocket Excavation	Partial excavation of the tunnel face in unstable ground conditions by which small areas (pockets) of ground are excavated immediately followed by shotcrete installation. A series of pockets are excavated following the drift shape allowing the installation of the shotcrete lining. Typically, a central face stabilization wedge remains in the face that is excavated either during the next excavation round in sequence or after completion of the full shotcrete lining installation.
Passive reinforcement	Reinforcing element that is not prestressed or tensioned artificially in the rock, when installed, i.e. rock dowel.
Pattern Reinforcement or Pattern Bolting	The installation of reinforcement elements in a regular pattern over the excavation surface.
Phreatic surface	That surface of a body of unconfined ground water at which the pressure is equal to that of the atmosphere.
Pillar	A column or area of coal or ore left to support the overlaying strata or hanging wall in mines.
Pilot drift or pilot tunnel	A drift or tunnel driven to a small part of the dimensions of a large drift or tunnel. It is used to investigate the rock conditions in advance of the main tunnel excavation, or to permit installation of ground support before the principal mass of rock is removed.
Pneumatically applied mortar or concrete	See shotcrete.
Portal	The entrance from the ground surface to a tunnel.
Pre-reinforcement	Installation of reinforcement in a rock mass before excavation commences.
Principal stress	A stress that is perpendicular to one of three mutually perpendicular planes that intersect at a point on which the shear stress is zero; a stress that is normal to a principal plane of stress. The three principal stresses are identified as least or minimum, intermediate, and greatest or maximum.

Raise	A shaft excavated upwards (vertical or sloping). It is usually cheaper to raise a shaft than to sink it since the cost of mucking is negligible when the slope of the raise exceeds 40" from the horizontal.
Ravening Ground	Poorly consolidated or cemented materials that can stand up for several minutes to several hours at a fresh cut, but then start to slough, slake, or scale off
Rebar Spiling	Reinforcement rebars installed at the tunnel heading ahead of excavation and grouted as a means to pre-support the ground. They can be installed in pre-drilled and grout filled holes or rammed into the soft ground
Recessed rock anchor	A rock anchor placed to reinforce the rock behind the final excavation line after a portion of the tunnel cross section is excavated but prior to excavating to the final line.
Reinforcement	Structural steel reinforcement improving the moment capacity of a concrete section.
Relievers or relief holes	The holes fired after the cut holes and before the lifter holes or rib (crown, perimeter) holes.
Retarder	Admixture for hydration control to delay setting of wet shotcrete.
Rib	1. An arched individual frame, usually of steel, used in tunnels to support the excavation. Also used to designate the side of a tunnel. 2. An H- or I-beam steel support for a tunnel excavation (see Set).
Rib holes	Holes drilled at the side of the tunnel of shaft and fired last or next to last, i.e., before or after lifter holes.
Road header	A mechanical excavator consisting of a rotating cutterhead mounted on a boom; boom may be mounted on wheels or tracks or in a tunnel boring machine.
Rock Anchor	Rock anchors are tensioned tendons anchored to the ground over a defined length.
Rock bolt	A tensioned reinforcement element consisting of a rod, a mechanical or grouted anchorage, and a plate and nut for tensioning by torquing the nut or for retaining tension applied by direct pull.
Rock dowel	An untensioned reinforcement element consisting of a rod embedded in a grout-filled hole and bonded to the surrounding ground along their entire length (fully bonded) either by friction or grout.
Rock mass	Ground mass built up by in situ pieces of rock material of which are limited by discontinuities. Properties controlled by grade of weathering, discontinuities, fillings, and orientation of discontinuities.

Rock reinforcement	Elements reinforcing a jointed rock mass to enhance the rock mass strength and reinforce the rock's natural tendency to support itself.. Passive (dowels, spiles) or active (bolts, anchors) elements are used. Rock mass reinforcement can be installed either in spot applications or systematically. The reinforcement elements used in SEM tunneling are typically steel or fiberglass bars or pipes in conjunction with shotcrete on the rock surface.
Rock support	The placement of supports such as wood sets, steel sets, or reinforced concrete linings to provide resistance to inward movement of rock toward the excavation.
Round	A group of holes fired at nearly the same time. The term is also used to denote a cycle of excavation consisting of drilling blast holes, loading, firing, and then mucking.
SINTEF	Foundation for Scientific and Industrial Research at the Norwegian Institute of Technology
Scaling	The removal of loose rock adhering to the solid face after a shot has been fired. A long scaling bar is used for this purpose.
Segments	Sections that make up a ring of support or lining; commonly steel or precast concrete.
Set	The temporary support, usually of Steel or timber, inserted at intervals in a tunnel to support. The ground as a heading is excavated (see Rib).
Shaft	An elongated linear excavation, usually vertical, But may be excavated at angles greater than 30 deg from the horizontal.
Shear	A deformation that forms from stresses that displace one part of the rock past the adjacent part along a fracture surface.
Shield	A steel tube shaped to fit the excavation line of a tunnel (usually cylindrical) and used to provide support for the tunnel; provides space within its tail for erecting supports; protects the men excavating and erecting supports; and if breast boards are required, provides supports for them. The outer surface of the shield is called the shield skin.
Shield tail (or skirt)	An extension to the rear of the shield skin that supports soft ground and enables the tunnel primary lining to be erected within its protection.
Shotcrete	Concrete applied through a nozzle by compressed air and, if necessary, containing admixtures to provide quick set, high early strength and satisfactory adhesion.
Shove	The act of advancing a TBM or shield with hydraulic jacks.
Skip	A metal box for carrying reek, moved vertically or along an incline.

Soft Ground	Deteriorated rock or residual soil with limited compressive strength and stand-up time.
Spall	A chip or splinter of rock. Also, to break rock into smaller pieces.
Spiles	Pointed boards or steel rods driven ahead of the excavation, (similar to forepoles).
Spoil	See muck.
Spot reinforcement or spot dowelling or bolting	Localized reinforcement to secure individual rock blocks and wedges in place. Spot reinforcement may be in addition to pattern reinforcement or internal support systems.
Spray Shadow	In shotcrete applications a shadow generated by objects (e.g. reinforcement, fixing devices). The shotcrete within this shadow area is less compacted and of low quality.
Spring line	The point where the curved portion of the roof meets the top of the wall. In a circular tunnel, the spring lines are at opposite ends of the horizontal center line.
Squeezing ground	Material that exerts heavy pressure on the circumference of the tunnel after excavation has passed through that area.
Stand-up-time	The time that elapses between the exposure of reek or soil in a tunnel excavation and the beginning of noticeable movements of the ground.
Starter tunnel	A relatively short tunnel excavated at a portal in which a tunnel boring machine is assembled and mobilized.
Struts	Compression supports placed between tunnel sets.
Systematic Rock Dowelling/Bolting	Rock reinforcement applied in a systematic pattern designed to suit the ground conditions expected.
TBM	Tunnel boring machine.
Tail void	The annular space between the outside diameter of the shield and the outside of the segmental lining.
Tie rods	Tension members between sets to maintain spacing. These pull the sets against the struts.
Tight	Rock remaining within the minimum excavation lines after completion of a round-that is, material that would make a template fit tight. "Shooting tights" requires closely placed and lightly loaded holes.
Timber sets	The complete frames of temporary timbering inserted at intervals to support the ground as heading is excavated.

Top heading	1. The upper section of the tunnel. 2. A tunnel excavation method where the complete top half of the tunnel is excavated before the bottom section is started.
Tunnel Boring Machine (TBM)	A machine that excavates a tunnel by drilling out the heading to full size in one operation; sometimes called a mole. The tunnel boring machine is typically propelled forward by jacking off the excavation supports emplaced behind it or by gripping the side of the excavation.
Tunnel Pre-support	Systematic measures including pre-spiling with bars or pipes, grouted pipe arch canopy or steel sheets installed from within the tunnel or prior to tunnel construction.
Water table	The upper limit of the ground saturated with water.
Waterproofing System	A layered system consisting of a drainage material (i.e. Geotextile) and a flexible, continuous synthetic membrane (typically PVC).
Weathering	Destructive processes, such as the discoloration, softening, crumbling, or pitting of rock surfaces brought about by exposure to the atmosphere and its agents.
Wet Mix	Mixture being supplied to the nozzle readily batched with water and admixtures.
Yield Element	Structural element of high deformation capability applied within the Initial Shotcrete Lining to facilitate controlled deformation.

Appendix H

Deficiency and Reference Legends for Tunnel Inspection

Appendix H
Deficiency and Reference Legends for Tunnel Inspection

H.1 Deficiency Legends

Exhaust Duct Hangers- Vertical & Diagonal

EDH1	Hanger is in good condition
EDH2	Hanger needs to be repaired
EDH3	Hanger needs to be replaced

Concrete Masonry Blocks

CMU1	Block - Loss Of Mortar
CMU2	Block - Cracked
CMU3	Block - Missing
CMU4	Block - Section Loss
CMU5	Block – Special

Concrete Cracks

C1	Concrete Crack < ⅛"
C2	Concrete Crack ⅛ - ¼"
C3	Concrete Crack ¼ - ½"
C4	Concrete Crack > ½"

Concrete Ceiling Panels

CCP1	Misaligned
CCP2	Bent
CCP3	Broken
CCP4	Buckled
CCP5	Joints Leak

Concrete Wall Panels

CWP1	Misaligned
CWP2	Tiles Cracked
CWP3	Tiles Broken
CWP4	Eye Bolts
CWP5	Tie down Bolts
CWP6	Longit. Stainless Steel Mount. Brkt

Other Codes

CD	Collision Damage
CLG	Clogged
COR	Corrosion
CR	Crack

Bolt Connections

B1	Surface Rust
B2	Loss Of Section %
B3	Out Of Plane
B4	Broken
B5	Buckled
B6	Other
B7	Missing
B8	Anchorage loose/creep

Other Codes

BAD	Bad
BAR	Bare
BEN	Bent
BKG	Blockage
BLN	Blown
BRK	Broken
BUC	Buckled Column

Other Concrete Cracking

H1	Hairline Cracking - Light
H2	Hairline Cracking - Medium
HORZ	Horizontal Crack
DFW	Diagonal Crack From Wall
LONGIT	Longitudinal Crack
MC1	Map Cracking - Nonrepairable
MC2	Map Cracking - Repairable
MJC	Mortar Joint Crack
PMCR	Previous Map Cracking
RC	Reflective Cracking
TFW	Transverse Crack From Wall
TRANS	Transverse Crack

Concrete Delaminations

D	Delamination

Other Codes

D	Dry (In "Moisture" column)
DAM	Damaged
DCOL	Discoloration
DEF	Deflection
DEI	Defective
DET	Deteriorated
DIR	Dirty
DIS	Disintegrated
DISC	Disconnected
DIST	Distorted
DS	Differential Settlement

Exhaust Duct Hangers – Vert/ Diag

DH1	Surface Rust
DH2	Loss Of Section %
DH3	Loss Of Tension
DH4	Out Of Plane
DH5	Broken
DH6	Buckled
DH7	Anchorage loose/creep

Other Codes

EFF	Efflorescence
EN	Excessive Noise
ER	Eroded
EV	Excessive Vibration
EXP	Exposed

Framing Steel

F1	Surface Rust
F2	Loss of Section %
F3	Out of Plane

F4	Broken
F5	Buckled
F6	Other
F7	Anchorage loose/creep

Steel Liner Plate Flanges

FL1	Surface Rust
FL2	Loss Of Section %
FL3	Out Of Plane

Glass Block Units

GB1	Joint Material Cracked or Missing
GB2	Cracked Block
GB3	Broken Block
GB4	Missing Block

Other Codes

GEN	General
HAZM	Hazardous Materials
HC	Honeycombing
HO	Hole

Encrustation

I1	Encrustation Light
I2	Encrustation Heavy

Other Codes

INCO	Inadequate Coverage
IV	Insufficient Ventilation

Concrete Joints

J1	Joint < ⅛"
J2	Joint ⅛" – ¼"
J3	Joint ¼ - ½"
J4	Joint > ½"
J5	Special Joint

Tunnel Lighting

LF1	Light Fixture Not Working
LF2	Light Fixture Casing Crcked or Brk
LF3	Light Fixture Mounting Bracket
LF4	Light Fixture Anchorage
LH	Loose Handle
LOC	Location (No Deficiency)
LOO	Loose

Tunnel Moisture

M1	Damp Patch
M2	Standing Drop
M3	Dripping
M4	Continuous Leak
PM	Past Moisture

Metal Ceiling Module Panels- Pre-fabricated

MCP1	Misaligned
MCP2	Bent
MCP3	Broken
MCP4	Buckled
MCP5	Joints Leak

Miscellaneous Metals
Framing Steel Suspended
Ceiling Support Assembly

MF1	Surface Rust
MF2	Loss of Section %
MF3	Out of Plane
MF4	Broken

Miscellaneous Metals (continued)
Framing Steel Suspended
Ceiling Support Assembly

MF5	Buckled
MF6	Other
MF6	Anchorage loose/creep

Miscellaneous Metals
Conduit Support Assembly

MS1	Surface Rust
MS2	Loss of Section %
MS3	Out of Plane
MS4	Broken
MS5	Buckled
MS6	Other

Other Codes

MI	Missing
MISAL	Misaligned
P	Ponding
PLG	Plugged
PR	Previous Repair

Paint

P1	Paint - Blister
P2	Paint – Peeling

Rebar

R1	Rebar-Surface Rust
R2	Rebar-Loss Of Section
R3	Rebar - Bent
R4	Rebar - Broken
R5	Rebar – Buckled
R6	Rebar – Special

Other Codes

RCJ	Recaulk Joint
RPJ	Repoint Joint
RPMJ	Repoint Mortar Joint
RUS	Rust

Concrete Spalls

S1	Spall < 2"
S2	Spall to rebar
S3	Spall behind rebar
S4	Special concrete spall

Steel Liner Plate Segments

SP1	Surface Rust
SP2	Loss Of Section %
SP3	Out Of Plane
SP4	Broken
SP5	Buckled
SP6	Other

Other Codes

SAG	Wire Mesh Sagging

Concrete Scaling

SC1	<1/4" Light Conc Scale
SC2	>1/4" Deep Conc Scale

Steel Rust

SR1	Steel Rust -Surface
SR2	Steel Rust - Pitting
SR3	Steel Rust - Section Loss
SR4	Steel Rust – Severe

Glazed Brick Or Block

FSP1	Minor Surface Spall, No Repair
FSP2	Major Surface Spall, Replace

Other Codes

SAG	Wire Mesh Sagging
ST	Stalactite/Stalagmite
VBC	Violation Of Code
VEC	Violation Of Electrical Code
VOSH	Violation Of OSHA
VPC	Violation Of Plumbing Code
VSHC	Violation Of State Health Code
WA	Warped
WD	Water Damage
WHO	Wires Hanging Out
WO	Worn

Sign Supports

SSP1	Minor rust
SS2	Loose components (elec/mech
SS3	Anchorage Loose/creep

H.2 Reference Legends

Code	Description	Code	Description
ABD	Automatic Ball Drip	DG	Diesel Generator (Emerg.)
ACP	Air Compressor	DH	Duct Hanger
ACU	Air Conditioning Unit	DIF	Diffuser
AF	Anchors & Fasteners	DMTR	Damper Motor
AH	Access Hatch	DPN	Distribution Panel
AHU	Air Handling Unit	DR	Door
AL	Alarm	DA	Damper
ALC	Air Lock Concrete	DRN	Drain
AP	Access Panel	DS	Disconnect Switch
AV	Air Vent	DSB	Distribution Switchboard
B	Beam Reinforced Conc.	DT	Drain trough - Safety walk
BAT	Battery	DW	Duct Work
BC	Battery Charger	ECP	Equipment Control Panel
BFP	Backflow Preventer	EJR	Ejector
BCR	Bituminous Concrete - Rwy	EL	Emergency Light
BL	Block – CMU and Glazed	EUH	Electric Unit Heater
BOI	Boiler	EW	Eye Wash
BRE	Breeching	EWC	Electric Water Cooler
BR	Brick– Includes Glazed	EX	Exit Light
BS	Beam - Steel (Not encased)	EXV	Exhaust Ventilator
C	Ceiling – Concrete	F	Floor - Concrete
CA	Cables	FA	Fire Alarm
CAH	Cabinet Heater	FAI	Fresh Air Intake
CAN	Canopy	FAN	Fan
CAP	Capacitor	FCT	Faucet
CB	Catch Basin	FDP	Fire Damper
CBC	Ceiling - Beam Conc Surface	FE	Fire Extinguisher
CBK	Circuit Breaker	FH	Fire Hose
CBR	Cross Bracing	FHC	Fire Hose Cabinet
CC	Cable Chase	FHV	Fire Hose Valve
CTSS	Cable Tray Support Steel	FIL	Filter
CCO	Column - Concrete	FL	Flue
CCP	Concrete Ceiling Panels	FLP	Flue Plate
CCPF	Concrete Ceiling Panel Flues	FLA	Flashing
CCTV	Closed Circuit Television	FLC	Flexible Connector
CD	Conduit – Embedded	FLCP	Fan Local Control Panel
CDE	Conduit – Exposed	FM	Force Main
CESB	Concrete Encased Steel Beam	FP	Fire Proofing – Spray On
CF	Cabinet Fan	FT	FUCO Tube
CFO	Column Foundation	GAU	Gauge
CMU	Concrete Masonry Unit	GB	Glass Block
CO	Concrete	GI	Girder - Concrete Encased
COM	CO - Monitor	GIS	Girder - Steel
CPL	Control Panel	GND	Ground
CST	Column - Steel	GR	Grout
CURB	Curb	GRA	Grating
CWP	Concrete Wall Panel	GRL	Grille
CWPB	Concrete Wall Panel Brac	GU	Gunite
		GUT	Gutter

Code	Description	Code	Description
HB	Hose Bib	**SIS**	Sign Supports
HC	Heating Coil	**SK**	Sink
HE	Heat Exchanger.	**SL**	Sleeve
HR	Handrail	**SM**	Stone Masonry
HSG	Housing	**SMW**	Light Gage Sheet Metal Walls – Exhaust Duct
HTC	High Tension Splicing Chmbr	**SNR**	Sensor
HTR	Heater	**SO**	Soil Pipe
HUM	Humidistat	**SPC**	Standpipe Cabinet
IC	Island Concrete – Toll Booth	**ST**	Stair
IS	Inlet Screen	**STK**	Stack
JB	Junction Box	**STP**	Steam Trap
JT	Joint - Construction/Expan	**STR**	Strainer
L	Leader	**STRC**	Strip Recorder
LA	Ladders	**SWC**	Safety walk - Concrete
LAV	Lavatory	**TB**	Toll Booth
LF	Light Fixture	**TBT**	Toll Booth Tunnel
LFS	Light Fixture Support	**TEL**	Telephone System
LI	Lintel	**TH**	Thermostat
LL	Light Level	**TOI**	Toilet Area
LS	Light Switch	**TS**	Traffic Signal
M	Miscellaneous Metal	**TSW**	Transfer Switch
ME	Meter	**TV**	Turning Vane
MF	Motor Foundation	**UH**	Unit Heater
MH	Manhole	**V**	Valve
MM	Motor Mount	**VB**	Vacuum Breaker
PA	Public Address	**VI**	Video System
PB	Pull Box (Electrical)	**VNT**	Vent
PBS	Push Button Station	**VS**	Ventilation Shaft
PI	Piping	**W**	Wall - Concrete
PLS	Steel Plates	**WAM**	Water Meter
PNL	Panel Board	**WB**	Wall - Block (CMU)
POP	Polymer Panels	**WBM**	Wall Beam
PP	Parapet	**WBR**	Wall - Brick
PT	Partition	**WC**	Water Closet
PV	Pavement	**WCB**	Wall - Cinder Block
RAS	Radio System	**WH**	Wall Hydrant
RCP	Receptacle	**WHA**	Water Hammer Arrestor
RLY	Relay	**WHL**	Wheel
RM	Roof - Membrane	**WI**	Window
RMP	Remote Monitoring Panel	**WIR**	Wire (Elect)
S	Structural Steel	**WL**	Window Louvers
SF	Shaft (Mech)	**WP**	Waterproofing
SH	Shaft (Misc.)	**WR**	Retaining Wall
SHE	Sheave	**WST**	Waste
SHFT	Elevator Shaft	**WT**	Wall - Tile
SHW	Shower	**XFR**	Transformer (Dry Type)
SI	Sign		

Design and Construction of Road Tunnels: Part 4 Obstacles and Mitigations Quiz

1. True or False: Tunnels, in general, perform better during earthquakes than above ground structures?

   - True
   - False

2. **Which of the following is not a step in analyzing seismic hazard?**

   - deterministic hazard analysis
   - Identification of the seismic sources capable of strong ground motions at the project site
   - Evaluation of the seismic potential for each capable source
   - Evaluation of the intensity of the design ground motions at the project site

3. **Identification of capable seismic sources together with evaluation of the seismic potential of each capable source may be referred to as_________.**

   - seismic source characterization
   - US Geological Survey
   - probabilistic seismic hazard evaluation
   - deterministic seismic hazard evaluation

4. **What are the steps in a deterministic seismic hazard analysis?**

   - Establish the location and characteristics (e.g., style of faulting) of all potential earthquake sources that might affect the site. For each source, assign a representative earthquake magnitude.
   - Select an appropriate attenuation relationship and estimate the ground motion parameters at the site from each capable fault as a function of earthquake magnitude, fault mechanism, site-to source distance, and site conditions.
   - Screen the capable (active) faults on the basis of magnitude and the intensity of the ground motions at the site to determine the governing source.
   - All of the above

Design and Construction of Road Tunnels: Part 4 Obstacles and Mitigations Quiz

5. **__________analysis incorporates the likelihood of a fault rupturing and the distribution of earthquake magnitudes associated with fault rupture into the assessment of the intensity of the design ground motion at a site.**

   - Probabilistic hazard
   - Deterministic hazard
   - Existing hazard
   - Seismic hazard

6. **What are the three main factors influencing tunnel seismic performance?**

   - Ground Shaking, seismic hazard, and ground failure
   - Geological conditions, tunnel construction, and body waves
   - Seismic hazard, geologic conditions, and tunnel design
   - Ground failure, traveling waves, and ground shaking

7. **In the event of a moderate to large magnitude earthquake, and an active fault crosses the tunnel alignment, there will be a hazard of ________ through the tunnel.**

   - [ ] landsliding
   - [ ] direct shearing displacement
   - [ ] differential deformation
   - [ ] liquefaction

8. True or False: Underground tunnel structures undergo three primary modes of deformation during seismic shaking: ovaling/racking, axial and curvature deformations.

   - True
   - False

Design and Construction of Road Tunnels: Part 4 Obstacles and Mitigations Quiz

9. **What deformation is caused primarily by seismic waves propagating perpendicular to the tunnel longitudinal axis?**

   - Curvature
   - Axial
   - Ovaling / Racking
   - Body waves

10. True or False: Cut-and-cover tunnels in soil tend to be more vulnerable than those excavated into rock because of the larger soil shear deformations causing the tunnel racking.

    - True
    - False

11. **According to OSHA, how much air should each person working underground be supplied with?**

    - 200 cubic feet per minute of air (cfm)
    - 100 cubic feet per minute of air (cfm)
    - 50 cubic feet per minute of air (cfm)
    - 300 cubic feet per minute of air (cfm)

12. **What is the series of individual activities that must be completed before the subsequent activities can start with an underground construction for tunnels that employ drilling and blasting to create the tunnel opening?**

    - drill, load, shoot, wait for fresh air, muck and support
    - cut, muck, and support
    - drill, cut, muck, support
    - support, muck, drill, load, shoot, and wait

Design and Construction of Road Tunnels: Part 4 Obstacles and Mitigations Quiz

13. **What term is used for material that has been excavated from tunnel construction?**

    - Muck
    - Debris
    - Earth
    - Aggregate

14. **There are numerous cost drivers associated with underground construction, what is the single most important driver of project cost?**

    - The ground through which the tunnel will be driven
    - The equipment used to drive the tunnel
    - The support elements
    - The safety of the workers

15. **What is a deep benchmark?**

    - Are steel pipes/casings drilled into stable strata – preferably sound bedrock – outside the advancing tunnel's zone of influence.
    - Plastic casings drilled inside the tunnel
    - How deep a tunnel is able to be drilled
    - None of the above

16. **What is Structural Monitoring Points?**

    - Survey points that are placed directly on the structures of concern
    - Mobilization of survey crews
    - Monitoring closure of the ground across either open excavations or mined tunnels.
    - Monitoring subsurface deformations around excavations when rapid monitoring is required or when instrumented locations are difficult to access for continued manual readings.

Design and Construction of Road Tunnels: Part 4 Obstacles and Mitigations Quiz

17. **What are In-Place Inclinometers?**

    ○ Used for monitoring subsurface deformations around excavations when rapid monitoring is required or when instrumented locations are difficult to access for continued manual readings.

    ○ Are aluminum or plastic casings drilled vertically to below the level of construction into a stable stratum and used to determine whether the surrounding ground, either rock or unconsolidated material, is moving laterally toward the excavation.

    ○ Are survey points that are placed directly on the structures of concern

    ○ Are used for obtaining almost real time data on movements in three dimensions when it is not feasible to continually mobilize survey crews to collect data.

18. **What inclinometer measures the vertical rather than the lateral movements of the instrumented structure.**

    ○ Tiltmeter

    ○ In-Place inclinometer

    ○ Conventional inclinometer

    ○ Horizontal inclinometer

19. **What does a blast seismograph measure?**

    ○ Measure the vibration waves generated by blasting then propagate through ground, soil, and structures.

    ○ Measure three components of ground motion

    ○ Are used to monitor ground motion at structures within the zone of influence

    ○ All of the above

20. True or False. The primary function of most instrumentation programs is to monitor performance of the construction process in order to avoid or mitigate problems.

    ○ True

    ○ False

Design and Construction of Road Tunnels: Part 4 Obstacles and Mitigations Quiz

21. True or False. The most significant problem in constructed tunnels is groundwater intrusion.

    - True
    - False

22. **For joints that move what grout is appropriate to use for sealing cracks?**

    - Cementitious grout
    - Particle grout
    - Chemical grout
    - Epoxy grout

23. True or False. Cracking is the most common defect found in concrete tunnel liners.

    - True
    - False

24. **What is "ringing" a hanger?**

    - Vibrate or ring like a bell after being struck
    - Verify hangers are in tension
    - Inspections are important to tunnel safety to check for structural stability
    - All of the above

25. **What is the most common way to prevent falling rock fragments from dropping onto the roadway?**

    - Scale (remove) loose rock on a periodic basis
    - Blast the tunnel until all rock fragments fall down
    - Placement of steel liner roof as shelter
    - Both A and C