Chapter 6
Design Considerations

6-1. Fundamental Approach to Ground Support Design

a. Underground design must achieve functionality, stability, and safety of the underground openings during and after construction and for as long as the underground structure is expected to function. There is no recognized U.S. standard, practice, or code for the design of underground structures. Many designers apply codes such as ACI's Codes and Practices for concrete design, but these were developed for structures above ground, not for underground structures, and only parts of these codes apply to underground structures.

b. Designers often approach tunnel design by searching for modes of failure that can be analyzed (e.g., combined bending and compression in a lining), then apply them to more-or-less realistic but postulated situations (loading of a lining). While bending and compression are applicable failure modes for linings, many other modes of failure must be analyzed. In principle, all realistic modes of behavior or failure must be defined; then means by which these can be analyzed and mitigated must be found.

c. Failure modes are modes of behavior that could be considered unacceptable in terms of hazard, risk to cost or schedule during construction, environmental effect, or long-term failure of function. For underground structures, failure of function means different things for different kinds of structures: a certain amount of leakage in an urban highway tunnel might be a failure of function, while for a rural water conveyance tunnel such leakage might be perfectly acceptable.

d. The five basic design steps are outlined below:

(1) The functional requirements are defined in a broad sense. They include all hydraulic and geometric requirements, ancillary and environmental requirements and limitations, logistics, and maintenance requirements.

(2) Collect geologic and cultural data including all information required to define potential failure modes and analyze them, field and laboratory data, and cultural data to define environmental effects and constraints. These data may include ownership of right-of-way, the possibility of encountering contaminants, and sensitivity of structures to settlements.

(3) Determine plausible and possible failure modes including construction events, unsatisfactory long-term performance, and failure to meet environmental requirements. Examples include instability problems or groundwater inflow during construction, corrosion or excessive wear of ground support elements, excessive leakage (in or out), and settlements that may cause distress to adjacent existing structures.

(4) Design initial and final ground supports. Initial support includes all systems that are used to maintain a stable, safe opening during construction. Final supports are those systems that need to maintain a functional opening for the design life of the project. Initial supports may constitute a part of the final supports, or they may be the final support (e.g., precast segmental liner installed behind a TBM).

(5) Prepare contract documents. This is the synthesis of all design efforts and may include provisions to modify construction procedures based on observations. The contract documents also contain all information necessary for a competitive bidding process, and means to deal with claims and disputes.

e. The following subsections describe functional requirements of tunnels and shafts, typical and not so typical modes of failure of tunnels and shafts, including corrosion and seismic effects. Selection and design of initial ground support are described in Chapter 7, and final lining selection and design in Chapter 9.

6-2. Functional Requirements of Tunnels and Shafts

Most USACE tunnels are built for water conveyance, either for hydropower, fresh water transport, or flood control. Underground hydraulic structures may include drop and riser shafts, inclines, tunnels, intakes, outlets, intersections, bifurcations, energy dissipators, venturi sections, sediment control, surge chambers, gates, and valves.

a. *Types of flow in underground hydraulic structures*.

(1) Flow in underground hydraulic structures will be either open-channel flow or pressurized flow. Pressurized flow is usually under positive pressure, but negative pressures can also be encountered.

(2) If it is desired to maintain gravity flow conditions in a tunnel, then the size and grades must be designed to accomplish this. Usually, the variable flow quantities and input pressures (minimums and maximums) are given and cannot be adjusted. In some cases, geologic conditions may limit adjustments to grade. On the other hand, it may be desired to generate pressurized flow, for example in a hydropower intake tunnel to spin the turbines, in which case size and grade are selected for that purpose. Trade-offs can be made between size and grade to determine whether pressurized or gravity flow will occur and which is more desirable for a specific facility.

(3) Short tunnels of 100 m (330 ft) or less can be driven level, but longer tunnels are usually constructed at a minimum slope of 0.0001 (0.01 percent) to facilitate drainage.

(a) *Open-channel (gravity) flow hydraulic structures*. In open-channel flow, the water surface is exposed to the atmosphere. This will be the case so long as the rate of flow into the structure does not exceed the capacity as an open channel. For a gravity flow tunnel with multiple input sources or changes in cross section or grade, various points along the alignment must be analyzed to ascertain the flow volume and velocity to make certain that this condition is met. Hydraulic jumps can form within open channels if the slope of the channel is too steep or the outlet is submerged. If the hydraulic jump has sufficiently high energy, damage to the structure can result. This condition should be avoided.

(b) *Pressurized structures*. When the flow rate exceeds the open-channel capacity of the structure, it becomes pressurized. This may be a temporary condition or may be the normal operating configuration of the facility. Cavitation occurs in flowing liquids at pressures below the vapor pressure of the liquid. Because of low pressures, portions of the liquid vaporize, with subsequent formation of vapor cavities. As these cavities are carried a short distance downstream, abrupt pressure increases force them to collapse, or implode. The implosion and ensuing inrush of liquid produce regions of very high pressure, which extend into the pores of the hydraulic structure lining. Since these vapor cavities form and collapse at very high frequencies, weakening of the lining results as fatigue develops and pitting appears. Cavitation can be prevented by keeping the liquid pressure at all points above the vapor pressure. The occurrence of cavitation is a function of turbulence in the water flow and increases with tunnel roughness and flow velocity.

b. Hydraulic controls.

(1) Hydraulic controls are placed in a flow channel to regulate and measure flow and to maintain water levels upstream of a section. Over the full length of a tunnel, a variety of flow conditions may exist in each of the segments. Discharge and flow depth are determined by the slope, geometry, and lining of a tunnel and by the locations of hydraulic controls such as gates, weirs, valves, intakes, and drop structures. Within each segment of a tunnel, the segment inlet or outlet can serve as the control section. Inlet control will exist when water can flow through a tunnel segment at a greater rate than water can enter the inlet. Headwater depth and inlet geometry determine the inlet discharge capacity. Segments of a tunnel operating under inlet control will generally flow partially full.

(2) Outlet control occurs when control sections are placed at or near the end of a tunnel segment and water can enter the segment at a faster rate than it can flow through the segment. Tunnel segments flowing under outlet control will flow either full or partially full. The flow capacity of a section flowing under outlet control depends on the hydraulic factors upstream of the outlet.

(3) Weirs are one form of hydraulic control commonly used to regulate and measure flow in open channels. Many variations in weir design exist, most of which are accompanied by their own empirical equations for the design of the weir. Weir equations and coefficients are found in most textbooks dealing with open-channel flow.

c. Transient pressures.

(1) Transient pressures are a form of unsteady flow induced whenever the velocity of moving water in a closed conduit is disrupted. Causes include changes in valve or gate settings, pump or power failures, lining failures, and filling of empty lines too quickly. One type of transient flow is known as water hammer. This phenomenon is a significant design consideration in water tunnels because of the structural damage that can occur with excessive high or low pressures. There are many other types of transient flows in tunnels that can be caused by unequal filling rates at different locations along the tunnel: air entrainment, air releases, and hydraulic jumps. For structural analysis, lower safety or load factors are used when designing for transient pressures.

(2) Transient pressure pulses arise from the rapid conversion of kinetic energy to pressure and can be

positive or negative depending on position with respect to the obstruction. Pressure pulses will propagate throughout a tunnel or pipe system being reflected at the ends and transmitted and reflected where cross sections change. The magnitude and propagation speed of a pressure pulse are determined by the elastic characteristics of the fluid and the conduit and the rate at which the velocity is changed. All other factors being equal, the more rapid the velocity change, the more severe the change in pressure.

(3) Transient pressures are managed by careful placement of surge tanks, regulated valve closure times, surge relief valves, or a combination of these methods.

(4) Transient pressures should be analyzed for each and every tunnel by the hydraulic engineering staff for use in the design of pressure tunnels. For preliminary use, a transient pressure 50 percent higher than the operating design pressure is often used.

d *Air relief*.

(1) Air that occupies an empty or partially filled tunnel can become trapped and lead to operating difficulties ranging from increased head loss and unsteady flow to severe transients and blowouts. Air can enter a tunnel system by entrainment in the water at pump inlets, siphon breakers, drop structures, and hydraulic jumps. It can also form when pressure and temperature conditions cause dissolved air to be released.

(2) Engineering measures to reduce air entrainment include thorough evaluation of drop structures under all foreseeable flow conditions, elimination of hydraulic jumps by reducing channel slopes or other means, and dissipation of flow vortices at inlets.

(3) Air entrapment can lead to increased head losses caused by a constricted flow cross section, and more significantly, severe transient pressures when trapped air is allowed to vent rapidly. Air entrapment at changes in tunnel cross sections are avoided by matching tunnel crown elevations rather than matching the inverts. Vents to the ground surface frequently are used for air pressure relief.

e. *Roughness*.

(1) The roughness of a tunnel lining relative to its cross-sectional dimensions is fundamental to the efficiency with which it will convey water. Tunnel excavation methods, geometry, and lining type affect flow capacity and play important structural and economic roles in water tunnel design. The allowable velocities in different kinds of water tunnels are restricted by potential cavitation damage depending on the liner material used, sediment deposit, and flushing characteristics.

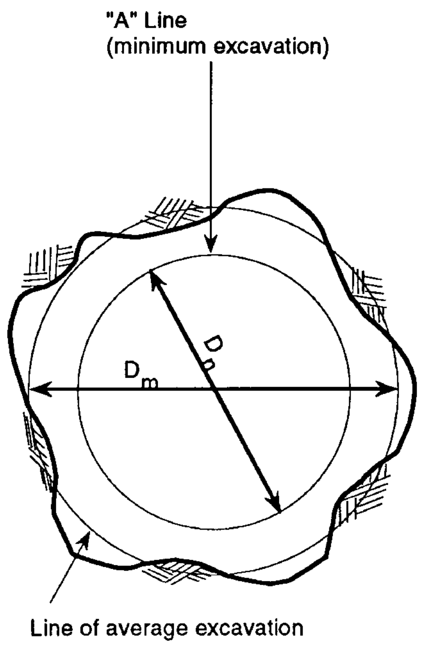
(2) The determination of tunnel friction factors for use with the Manning or Darcy-Weisbach flow equations is complicated by changes in flow depth, irregular channel geometries, and the wide range of roughnesses that occur when multiple lining types are used. Friction coefficients for the Manning and Darcy-Weisbach equations are each affected, but to different degrees, by changes in velocity, depth of flow, lining material, tunnel size, and tunnel shape. The Darcy-Weisbach approach is technically the more rigorous of the two equations; however, the Manning equation survives in practice because of its reasonable accuracy as an approximation for typical tunnel sizes and its relative simplicity.

(3) In practice, fluid velocities are limited so that turbulent conditions and the possibility of damage to the structure are limited. Velocities of less than about 3 m/s (10 ft/s) are considered safe in tunnels with no lining. Velocities between about 3 and 6 m/s (10-20 ft/s) usually necessitate concrete linings. For velocities greater than 6 m/s (20 ft/s), the risk of cavitation increases, and special precautions like steel or other types of inner lining must be taken to protect the inside of the structure. Where the water will carry sediments (silt, sand, gravel) the velocity should be kept below 3 m/s (10 ft/s).

(4) A study on friction losses in rock tunnels by Westfall (1989) recommends friction factors (Manning's roughness coefficient, *n*) for different excavation methods and lining types as follows:

Drill and blast excavation, unlined	$n = 0.038$
Tunnel boring machine excavation, unlined	$n = 0.018$
Lined with precast concrete segments	$n = 0.016$
Lined with cast-in-place concrete	$n = 0.013$
Lined with steel with mortar coat	$n = 0.014$
Lined with steel (diam > 3 m (10 ft))	$n = 0.013$
Lined with steel (diam < 3 m (10 ft))	$n = 0.012$

(5) Factors that can adversely affect friction include overbreak and rock fallout in unlined tunnels, misalignment of precast segments and concrete forms, sediment, and age. Westfall (1989) emphasizes the value of presenting several tunnel diameter and lining alternatives in final contract documents. Huval (1969) presents a method for computing an equivalent roughness for unlined rock tunnels that is employed for different tunnel stretches in an example by Sanchez-Trejo (1985). Figure 6-1 shows the basic



$$\text{Overbreak} = \frac{K}{2} = \frac{D_m - D_n}{2}$$

$$\text{Relative Roughness} = f_m = \left(\frac{1}{2 \log (D_m/K) + 1.14} \right)^2$$

$$\text{Manning's} \quad n = \sqrt{\frac{f_m D_m^{1/3}}{185}}$$

Figure 6-1. Roughness factor calculations for unlined tunnels

equations utilized by this method. Manning's *n* for composite linings of different roughness can be estimated as a weighted average of the friction factors for each surface where length of wetted perimeter of each surface is used for weighting. Figure 6-2 illustrates the variation in friction factor versus flow depth in a shotcrete-lined tunnel with a concrete-paved invert.

f. Drop shafts for vertical conveyance. Drop shafts are used in water conveyance tunnels to transfer flows from a higher elevation to a lower elevation. Such drop shafts are typically used in flood control and CSO systems. Drop shafts should be designed to dissipate the energy increase associated with the elevation drop; to remove any air that mixes or entrains with the water as it descends; and to minimize hydraulic head losses when the tunnels are surcharged.

(1) *Drop shaft components*. A drop shaft has three essential elements: an inlet structure, a vertical shaft barrel, and a combination energy dissipator and air separation chamber. The inlet structure's function is to provide a smooth transition from horizontal flow to the vertical drop shaft. The drop shaft barrel then transports the water to the lower elevation and in the process dissipates as much energy as possible. At the bottom of the drop shaft, a structure must be provided that will withstand the impact forces, remove any entrained air, and convey the water to the tunnel.

(2) *Basic consideration in drop shaft design*. Several factors must be considered in the design of drop shafts. These factors are variable discharge, impacts on the drop shaft floor, removal of entrained air, and head loss associated with the drop shaft. The selection of an appropriate drop shaft for a particular use involves determining which of these factors are most important. When the difference in elevation between the upper level flows and the tunnel is small, impacts on the drop shaft floor may be alleviated with a simple plunge pool. As the difference in elevation increases, removal of entrained air is necessary and floor impact becomes more severe. In cases where the tunnel hydraulic gradient can rise all the way up to the hydraulic gradient for the upper level flows, head loss also becomes a critical factor.

(3) *Variable discharge*. A drop shaft may be operated for steady-state flows, only during storm discharge periods, or as a combination of the two. The flow variability of a drop shaft has a considerable influence on the design. For instance, for steady-state flow the water surface elevation in the tunnel may be below the base of the drop shaft. In that case, a plunge pool is required at the drop shaft floor to dissipate energy. A shaft that handles only storm flows will not normally require a plunge pool because the water surface in the tunnel will submerge the drop shaft base and cushion the impacts.

(4) *Impact on the drop shaft floor*. The impact of the water on the floor of the drop shaft can be high, and steps should be taken to minimize it. This is accomplished by forcing a hydraulic jump within the shaft, by increasing the energy dissipation due to wall friction as the water descends, by entraining sufficient air to cushion the impact, or by providing a plunge pool at the bottom of the shaft. The plunge pool may be formed by a depressed sump or by the use of a weir located in the chamber at the base of the shaft and downstream of the shaft barrel. The required depth of the plunge pool can be determined by the use of the Dyas formula:

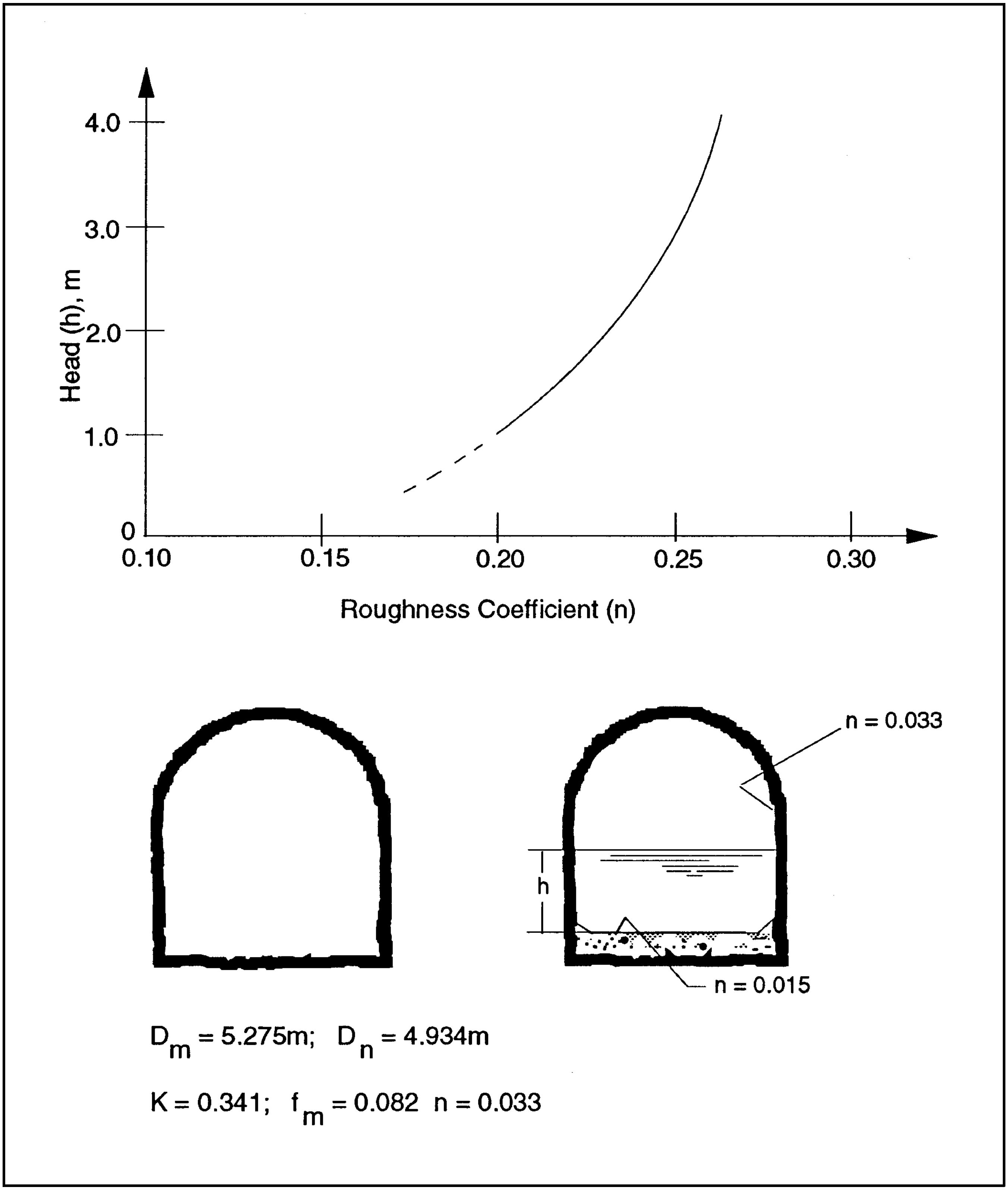


Figure 6-2. Friction factors for composite lined tunnel (see Figure 6-1 for definition of symbols)

$$Depth = 0.5h^{1/2}d_c^{1/3} \tag{6-1}$$

where

h = height of drop, ft

d_c = critical depth in inlet, ft

(5) *Removal of entrained air*. As the water falls through the drop shaft, it entrains, or mixes, with air. There are several advantages and disadvantages associated with air entrainment. The advantages are as follows:

- Presence of air minimizes the possibility of subatmospheric pressures and thus negates the harmful effects of cavitation.

- Impact of the falling water on the drop shaft floor is reduced by the cushioning effect of the air entrained in the water.

Disadvantages of air entrainment are as follows:

- Flow volume is bulked up and requires a larger drop shaft.

- In order to prevent the formation of damaging high-pressure air buildups, entrained air must be removed before entering the tunnel.

(6) *Head loss associated with the drop shaft*. Under certain conditions the tunnel hydraulic gradient may rise to levels equal to those of the upper level inflow. In these circumstances, the head losses become important because a large head loss may cause severe flooding in the upper level flow delivery system. For example, if this upper level delivery system is a sewer, large drop shaft head losses will result in flow backups into streets and/or basements.

(7) *Types of drop shafts*. Various types of drop shafts have been designed and constructed based on hydraulic laboratory model studies. Drop shafts as deep as 105 m (350 ft) have been constructed. The smaller structures, normally used for drops of less than 21 m (70 ft), are divided into several categories. These categories are drop manholes, vortex, morning glory, subatmospheric, and direct drop, air entraining.

(8) Drop manholes. Drop manholes are generally used in local sewer systems to transfer flows from a higher sewer to a lower sewer. These drop shafts are designed to minimize turbulence, which can release odorous gases and damage the shaft. A typical design has a personnel access upstream of the shaft that allows maintenance personnel to enter the lower sewer without climbing down the wet shaft.

(9) Vortex drop shafts. Flow enters the vortex-flow drop shaft tangentially and remains in contact with the drop shaft wall, forming a central air core as it descends. Since the flows through the inlet are spun against the shaft wall, the entry conditions are relatively smooth. Vortex drop shafts are effective for a wide range of discharges. The air core helps to evacuate the entrained air and to provide near atmospheric pressure throughout the shaft, so as to prevent any cavitation. Vortex drop shafts generally entrain less air than other types of drop shafts for two reasons. First, the flows are highly stable due to the entry conditions. Second, a reverse flow of air occurs in the core of the vortex, which causes much of the air entrained in the flow to be released and recirculated in the zone above the hydraulic grade line. Below the hydraulic grade line, the helical flow has a pressure gradient, which forces bubbles to move toward the center of the drop shaft where they are able to rise against the relatively slower moving water. Therefore, most air entrained by the flow is allowed to dissipate before it enters the tunnel. As the flows are spun against the walls of the drop shaft, significant energy is dissipated before the flow reaches the floor of the drop shaft. The dissipation is a consequence of the wall friction as the flows spiral down at high velocity. The remainder of the energy is dissipated in the air separation chamber by either a plunge pool or by the formation of a hydraulic jump. Several inlet configurations have been adopted to create a vortex flow down a drop shaft (see Figure 6-3). Based on various model studies, a vortex drop shaft is highly efficient when the turned gradient does not approach the level of the upper incoming flow. It is a good energy dissipator and has a high air removal rate.

(10) *Morning glory drop shafts*. Morning glory drop shafts employ a circular crested inlet structure. They are often used as outflows for reservoirs. Model studies have determined that the flow characteristics are controlled by three conditions: weir control, orifice control, and differential head control. The capacity of the morning glory drop shaft is limited by the size of the circular crest. No cavitation is expected in this type of drop shaft. Induced head losses could occur if the circular crest is inadequately designed. The U.S. Bureau of Reclamation recommends that the outlet tunnel be designed to flow 75 percent full to eliminate instability problems.

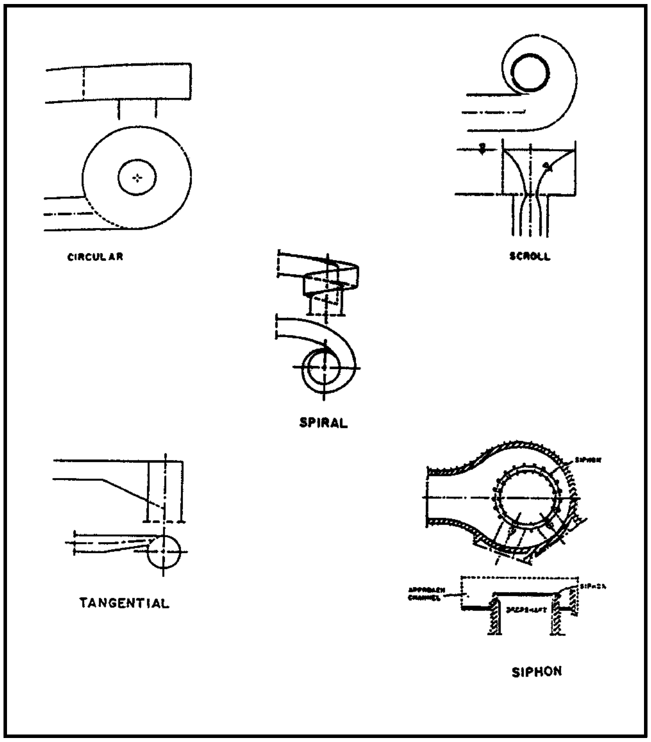


Figure 6-3. Five types of inlets for vortex-flow drop structures

(11) *Direct drop air entraining drop shafts.*

(a) Flow enters these drop shafts radially and descends through the shaft. The shaft diameter is designed to flow full with air entrained in the water, bulking it up enough to fill the drop shaft. The air entrained also provides a cushion for the water, reducing the floor impact. A large separation chamber is used at the base of the shaft and an air vent is necessary to allow the air to vent before entering the tunnel. This type of structure is very effective in dissipating energy and removing entrained air.

(b) Two types of direct-drop air entraining drop shafts are discussed below. The first of these consists of a sump chamber with a sloping top, as shown in Figure 6-4. The air vent is located inside of the drop shaft downcomer barrel, separated by a vertical slotted wall. The slots in the wall allow air to be recirculated into the falling water in the drop shaft resulting in the reduction of large air slugs and providing a more homogeneous mixture of air and water.

(c) At the bottom of the shaft is the sloped-roof air separation chamber. As the air is released from the mixture, it follows the sloping wall of the air collector back up to the air vent side of the vertical shaft and rises to the surface, some of it being recirculated through the slots into the drop shaft. If the drop shaft is to be used for steady-state flows, a plunge pool is built directly beneath the shaft barrel to dissipate the energy.

(d) This structure requires a rather large air separation chamber. For larger drop shafts, this requires a high chamber roof. During the design of the TARP system (Chicago) in rock, it was determined that this type of shaft was economical up to shaft diameters of 2.7 m (9 ft) with a maximum discharge capacity of 17 m^3/s (600 cfs).

(e) Another drop shaft design is suitable for drop shafts larger than 2.7 m (9 ft) in diameter. This drop shaft shown in Figure 6-5 has a separate shaft for the air vent downstream from the downcomer and connected to the downcomer above the crown of the incoming pipe. The air separation chamber has a horizontal roof. The air vent recycles air into the downcomer. This design can be used in much larger drop shafts, up to 6 m (20 ft) in diameter with a maximum discharge capacity of 127 m^3/s (4,500 cfs).

(f) Both structures handle a wide range of discharges and have head losses only one-fifth of those for vortex type shafts. These shafts are the only commonly used drop shafts that can adequately handle variable discharges, impacts on drop shaft floors, remove entrained air, and have minimum head losses to prevent backflow problems when tunnel gradients reach the levels of incoming flows.

(g) The large dimensions of both of these types of drop shafts, particularly the air separation chambers, necessitate mining a major chamber in rock with attendant rock reinforcement and lining. Larger sized versions of these drop shafts can be overexcavated and used as construction shafts.

g. Air removal. High-velocity streams of water may entrap and contain large quantities of air. Air entrainment causes the flow to be a heterogeneous mixture that varies in bulk density throughout the flow cross section and exhibits pulsating density variations.

(1) *Potential problems.*

(a) The engineer should eliminate the harmful effects brought about by the formation of high-energy hydraulic jumps within the tunnel; transient phenomena induced by rapid filling of the downstream end of a tunnel without

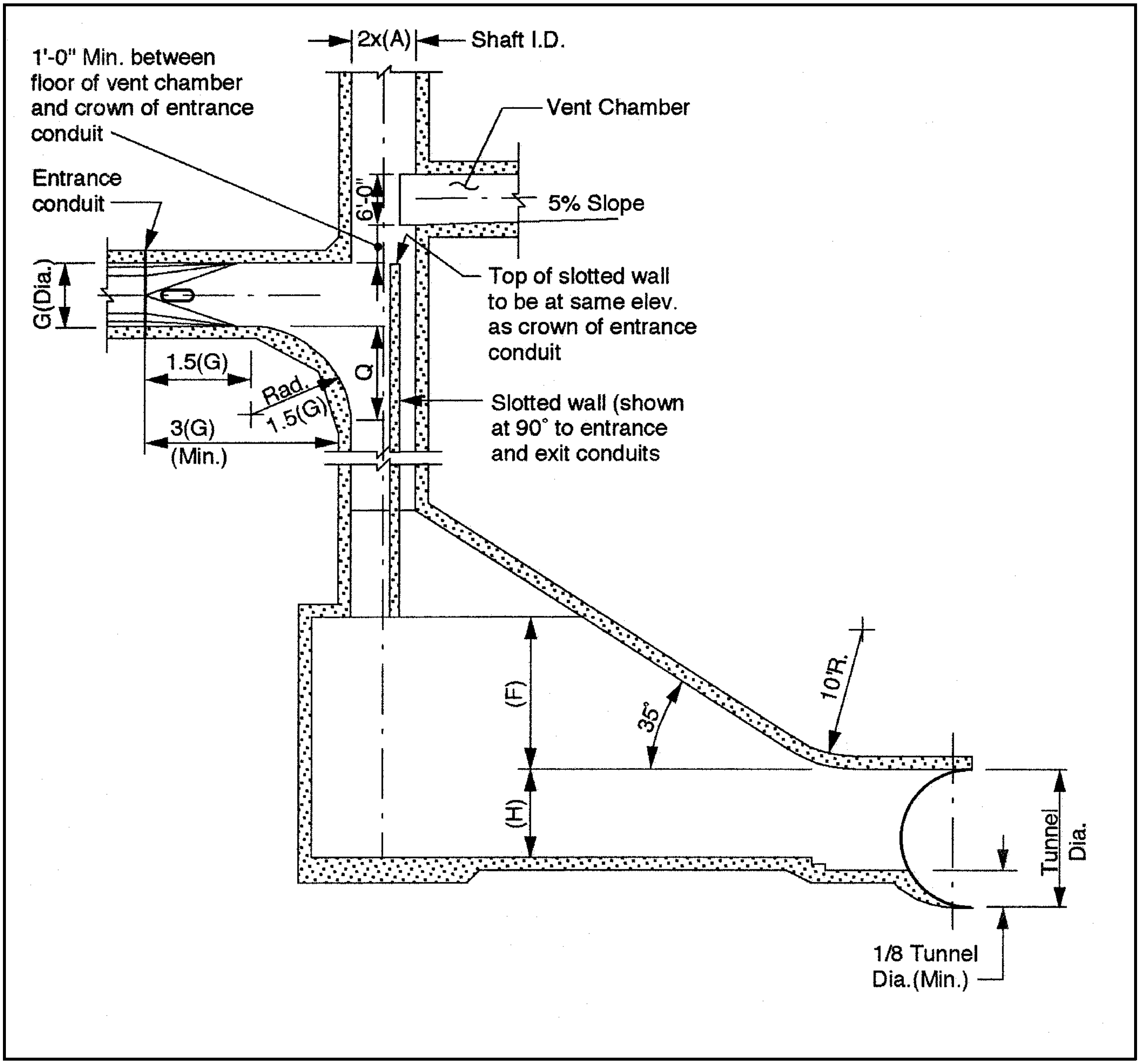


Figure 6-4. Direct-drop air entraining drop shaft

provisions for adequate surge shafts; the formation of air traps within the tunnel system; the introduction of entrained air into the tunnel from drop shafts; and the formation of vortices, which may enter the tunnel through shafts. In addition, the design should provide for the easy egress of air from a tunnel while it fills with water. Improper design can lead to one or more of the following phenomena, which may lead to structural damage:

- Blowbacks—high-pressure releases of air and water in the opposite direction of the flow.
- Blowouts—high-pressure releases of air and water in the same direction of the flow.
- Geysering—air/water venting above the ground surface through shafts located at any point along the tunnel.
- Transient and surging flows causing rapid dynamic instability and possible tunnel collapse.

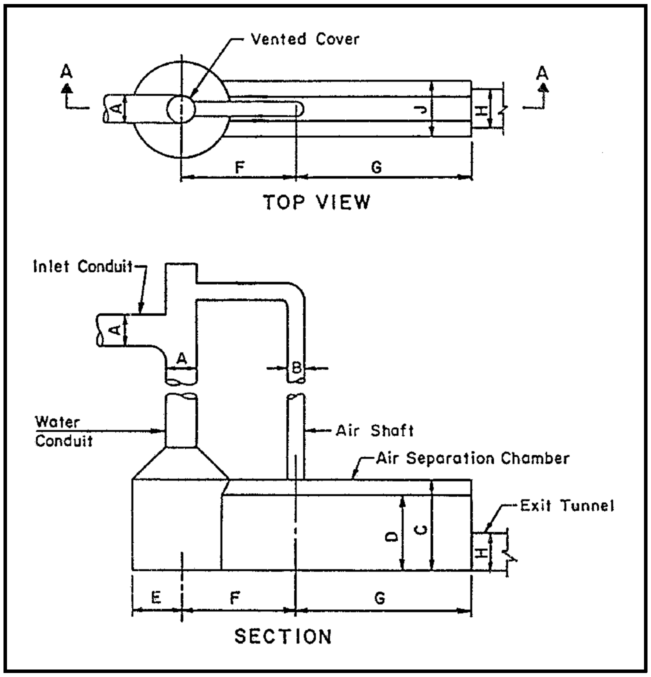


Figure 6-5. Direct-drop air entraining drop shaft with separate air vent

(b) As long as the depth downstream of a hydraulic jump does not reach the tunnel crown, jumps within tunnels are not a severe problem. When the downstream depth seals against the roof of the tunnel, the shock effects of air trapped downstream of the jump can create violent impacts and associated damage. High-energy hydraulic jumps have caused both blowouts and blowbacks. These rapidly escaping air pockets result in water rushing in to fill the voids, creating loud noises and pressure waves, which have resulted in stripping the lining from tunnels and shafts, partial tunnel collapse, and severe erosion.

(c) Even without the formation of hydraulic jumps, blowbacks, blowouts, and geysering, dynamic instability due to transients can take place whenever the downstream end of a tunnel is filling rapidly while air trapped within the system cannot escape at a reasonable rate. When the pressurization surge reaches an upstream end of the tunnel during the filling process, water will rise rapidly in shafts near the upstream end. Water levels in other shafts will also rise as the surge reflected by the upstream end travels downstream.

(d) In pressure tunnel flows, an air void can form at a bend connecting a vertical shaft to a horizontal tunnel. A sudden reduction in the flow rate can cause this void to vent back up the shaft and cause geysering.

(e) Inlet No. 2 of the Oroville Dam Diversion Tunnels experienced the development of vortex. The vortex grew in size and strength as the reservoir filled during the December 1964 flood. After the flood, the tunnel was dewatered and inspected throughout its entire length. Although the observed damage was relatively minor, it did consist of many rough scoured surfaces throughout the entire tunnel length.

(2) *Solutions*. The above-mentioned problems can be prevented by proper precautions during design. The following steps should be taken:

(a) Check the tunnel slopes for the development of supercritical flow and calculate whether a hydraulic jump can occur for any conceivable discharge. A hydraulic jump may not occur during the maximum design discharge but can occur for some lesser discharges. The tunnel slopes should be reduced if the check shows the potential for a hydraulic jump.

(b) Provide surge shafts of diameters at least equal to the diameter of the tunnel at both upstream and downstream ends of the tunnel. A transient analysis should be made during the design phase to determine how high these surge shafts should be.

(c) Whenever branch tunnels or drop shaft exit conduits meet another tunnel and whenever a tunnel changes diameter, always match tunnel crowns rather than inverts, to prevent the formation of air pockets.

(d) Prevent entrained air from entering the tunnel from drop shafts.

(e) Provide a splitter wall to suppress the development of vortices in the inlet to tunnels whenever it is apparent that strong vortex development may occur.

(f) Provide some form of inlet control to regulate or completely shut off all flows into each inlet tributary to the tunnel. This may usually be accomplished by the use of remotely controlled gates at each shaft inlet.

h. Control of infiltration and exfiltration.

(1) The phenomena of infiltration and exfiltration are of critical importance to water conveyance tunnels. Infiltration during construction should be reduced to acceptable levels in all types of tunnels. Significant infiltration after a water conveyance tunnel is completed is unacceptable. Inflows can cause loss of ground into the tunnel and result in surface settlements and damage to neighboring structures. The inflows may cause the adjacent groundwater table to be seriously lowered with resulting adverse impacts on water supply, trees, and vegetation. In flood control tunnels, groundwater infiltration can reduce the carrying capacity available to handle peak flows. Infiltration in water supply tunnels may lead to pollution of the supply. In sewer tunnels, infiltration contributes to increased water reclamation and pumping costs.

(2) Exfiltration from water conveyance tunnels also has potential for undesirable effects. In flood control and sewer tunnels, exfiltration may cause pollution of the adjacent groundwater. Exfiltration from water supply and power tunnels can result in serious reductions in available drinking water and energy supplies as well as revenue loss.

(3) The extent to which infiltration and exfiltration should be reduced must be determined before the design of the tunnel commences. It may be appropriate to apply different standards of water tightness to different sections of the tunnel. It is common practice to specify in the contract documents permissible inflows both during and after the construction of water conveyance tunnels.

i. Lake taps and connection to live tunnels. Connecting a new water conveyance tunnel to an existing high-pressure water tunnel or tapping a lake or reservoir is a task that requires careful advance planning. Obviously such connections are best made in the dry, but in certain cases this is not economically feasible. The following discussion highlights some alternatives.

(1) *Cofferdam.* For tunnels that are to connect to a relatively shallow lake, a ring cofferdam can be constructed from tunnel level below the bottom of the lake to an appropriate elevation above the water surface. The enclosed area can then be dewatered in order to make the connection between the lake and the future shaft and tunnel in the dry.

(2) *In-line tunnel diversion.* To connect a new tunnel to a live high-pressure tunnel, an in-line diversion pipe or series of pipes can be installed within the existing tunnel after it has been temporarily dewatered. A flow cutoff must then be installed around the in-line diversion pipes on both sides of the proposed connection to prevent water from flowing along the backs of the pipes into the connection. With the in-line diversion in place, the new tunnel connection can be made in the dry while the existing tunnel is fully pressurized. When the connection is completed, the existing tunnel may be dewatered again and the diversion pipes and cutoffs removed and the project completed.

(3) *Open-piercing method, lake taps.*

(a) The method is restricted to the construction of a connection in rock. In this method, the new tunnel is advanced as close to the existing high-pressure source as possible, leaving a rock plug in place above the tunnel crown. The tunnel near the connection should be constructed such that, when filled with water, a compressed air cushion will be created below the plug. This air cushion should be maintained until the final connecting blast is made. A rock trap is provided in the invert of the new tunnel below the plug. A shaft from ground surface to the new tunnel invert is also required as close as possible to the connection. A gate is provided on the side of this shaft furthest from the rock plug to seal any water from entering the tunnel beyond the shaft-rock plug section. The rock plug is then drilled and prepared for blasting to make the final connection. Next, the gate is closed and the tunnel (on the rock plug side of the shaft) and the shaft are filled with water to a depth slightly below the water level in the live tunnel or lake to be tapped. At this point, the air cushion below the plug should be checked for adequacy by remote monitoring and additional air pumped in if necessary. The charge is then detonated and the air cushion below the plug interrupts the water column to dampen the pressure shock and prevent damage to the new tunnel. Since the water pressure at the time of the blast is less inside the newly constructed tunnel, most of the rock blasted in the connection will collect in the rock trap. In this procedure, the final connection is left unlined.

(b) There are several other methods to execute lake taps. In 1988, the Alaska District employed the “dry method” for a lake tap for the Snettisham project near Juneau, Alaska. The final plug was about 3.3 by 3.3 by 3.6 m (11 by 11 by 12 ft) and blasted using a double burn-hole cut pattern. A buffer was made of a large plug of ice. Two rock traps were employed.

(c) The design and construction of lake taps and other high-pressure taps must be carried out with the help of specialists experienced in this type of work.

j. *Other requirements*. The hydraulic requirements of underground structures are of primary importance to design and construction. Other secondary considerations are listed below.

(1) *Construction tolerances*. With open-channel flow, tunnel grade elevation must be established with some precision to maintain the hydraulic properties of the facility. Accurate grade also provides better drainage during construction and avoids accumulation of water in depressions during construction. Grade tolerance for the finished tunnel is usually set at ±13 mm (0.5 in.) for relatively short tunnels, ±25 mm (1.0 in.) for large tunnels. A greater tolerance is given, for constructibility reasons, to tunnels lined with one-pass concrete segments. The centerline tolerance for the finished cast-in-place tunnel is often set at ±25 mm (1.0 in.). However, this tolerance is often irrelevant for functional purposes, and a much greater horizontal tolerance, up to ±150 mm (6 in.) or more can usually be accepted. For a cast-in-place lining, the tolerance on the inside diameter can be set at 0.5 percent, provided the lining thickness is not less than designated. For a precast segmental one-pass lining, a maximum out-of-roundness of 0.5 percent is usually acceptable. Surface irregularities should be kept below 6 mm (0.25 in.).

(2) *Unlined sections may need rock traps*. If a tunnel or shaft is unlined and may collect small pieces of rock or debris, traps are recommended to collect the debris so that it has a minimal effect on flow area, velocities, and friction losses, and so that it will not enter turbines or valves.

6-3. Modes of Failure of Tunnels and Shafts

It is convenient to distinguish between modes of failure that occur during construction and those that occur sometime during the operating life of the structure. Some failure mechanisms observed during construction may be present throughout the operating life if not properly controlled. Some construction failure modes were discussed in the earlier subsection on tunneling hazards (flooding, gases); others more related to the mechanics and chemistry of rock masses are discussed in this subsection. This discussion is not exhaustive because combinations of natural forces and the effects of construction can lead to events that cannot readily be categorized. Nonetheless, an understanding of the forces of nature working in a tunnel environment is helpful in preparing for design work. Failures of tunnels and shafts range from collapse or complete inundation with water and silt to merely disfiguring cracks. They all have underlying causes, and if these causes are understood, the potential exists to discover them ahead of time and prevent or prepare for them. The first set of examples of failure modes are encountered primarily during construction, but some of them may apply to finished tunnels left unlined or with insufficient ground support. The second series of examples apply to finished, lined tunnels. Failure of environmental nature, such as detrimental groundwater drawdown or damage due to settlements are discussed in Section 5-14.

a. Tunnel and shaft failure modes during construction.

(1) *Failures controlled by discontinuities*.

(a) Rock masses are usually full of discontinuities, bedding planes, fractures and joints, or larger discontinuities, faults, or shears that may form zones of weakness. These are planes of weakness where the rock mass may separate or shear during excavation. Whether or not they will separate or shear and cause a rock fall into the tunnel is largely a matter of geometry, and of the tensile and shear strength of the discontinuity.

(b) The tensile strength across a bedding plane is often poor or nonexistent. The shear strength, however, can be close to that of the adjoining materials, depending on the normal stress across the plane, as well as joint roughness and other surface characteristics. Because the excavation of a tunnel results in a general unloading of the tunnel environment, the shear strength of a bedding plane is often greatly reduced, depending on the orientation of the bedding plane relative to the opening. Therefore, bedding planes often participate in forming blocks of rock that can fall from tunnel roof, wall, or face.

(c) Shaley beds in a sandstone or limestone formation may appear to be sound at first exposure, but the unloading due to excavation combined with access to air and water can soften and cause slaking in such beds in hours or days such that they lose most of their tensile and shear strength and participate in the formation of rock falls. It is common in such bedded formations to experience rock falls days after excavation.

(d) Joints and fractures have no tensile strength, unless they have been healed by secondary deposition of minerals. The shear strength of a joint depends on a number of factors: Width, infilling (if any), local roughness, waviness on a larger scale, the strength of the joint wall (affected by weathering), and the presence of water.

(e) One discontinuity across or along the tunnel cannot form a block that will fall from the roof, wall, or face.

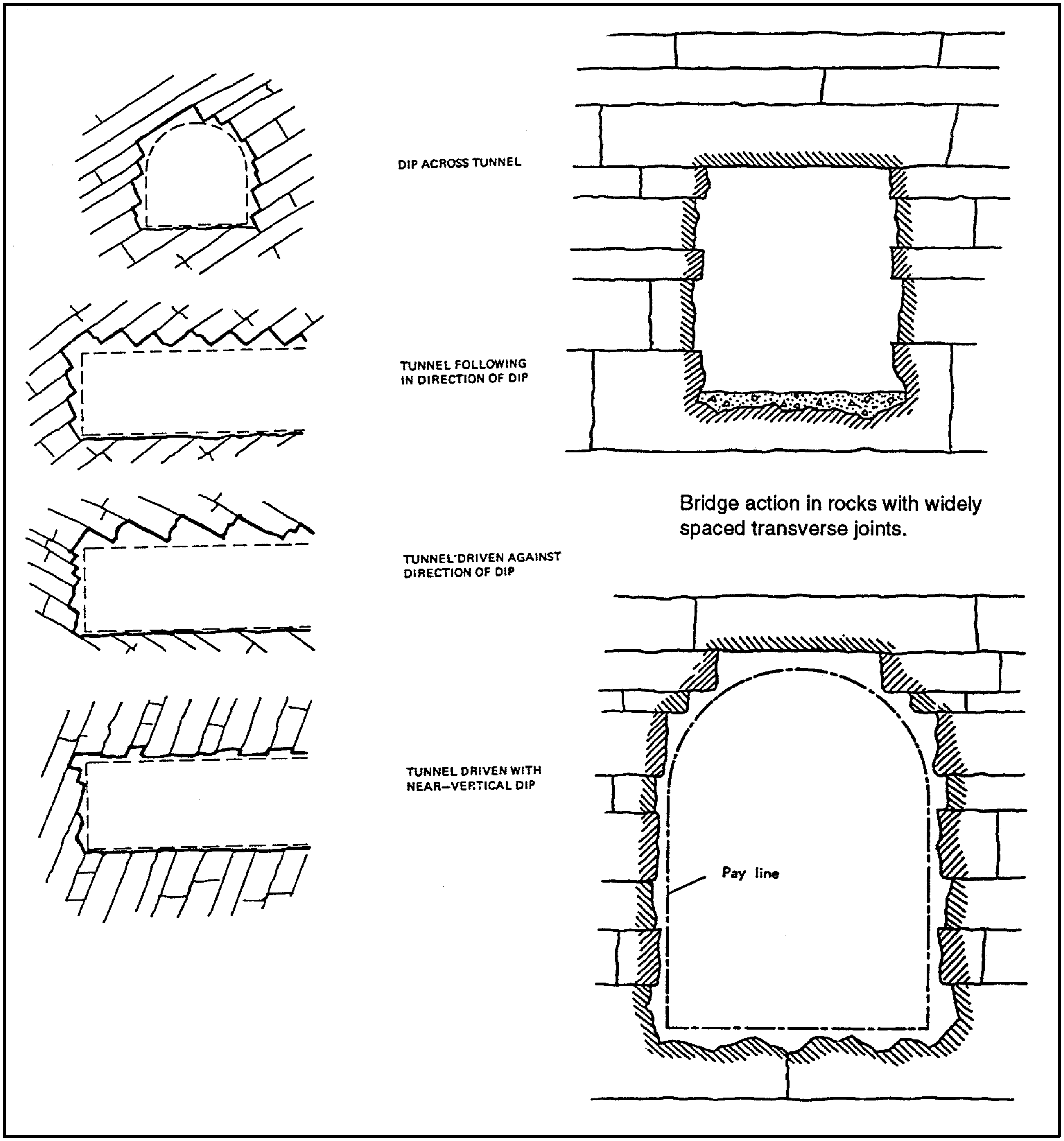
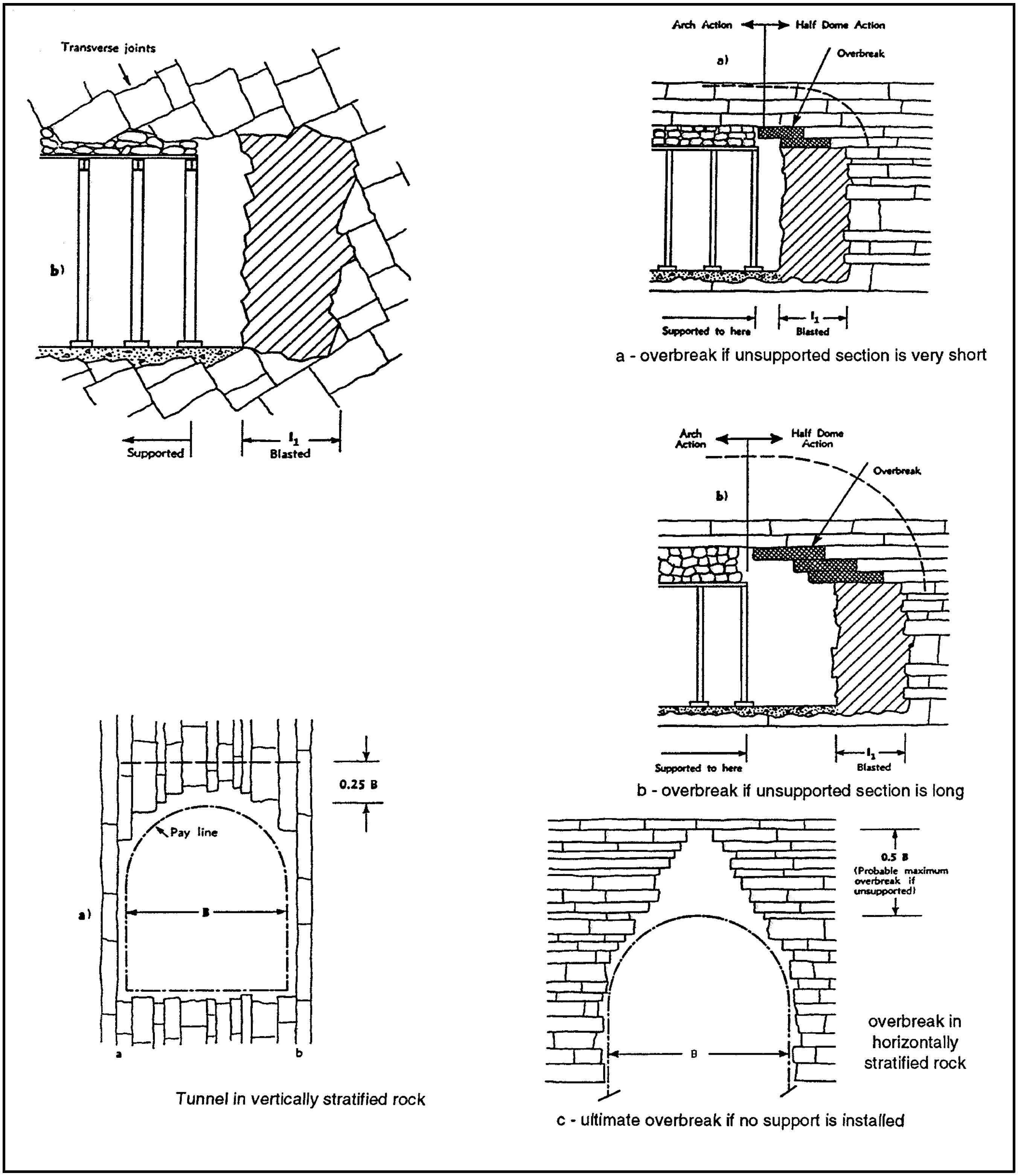


Figure 6-6. Examples of discontinuities (in part after Proctor and White 1946) (Continued)

It usually takes three intersecting discontinuities to form a loose block. However, gravity can help cause a cantilevered block to fail by bending or tension, and stress concentrations around the opening can result in other unfavorable fractures through intact rock, causing rock falls even with only two (sets of) discontinuities. Figure 6-6 shows several examples of how fractures and bedding planes can affect tunnel stability.



a - overbreak if unsupported section is very short

b - overbreak if unsupported section is long

Tunnel in vertically stratified rock

c - ultimate overbreak if no support is installed

Figure 6-6. (Concluded)

(f) If orientations and locations of discontinuities were known before tunnel construction, stability of blocks could in theory be predicted using graphic techniques or block theory (Goodman and Shi 1985). For a long tunnel, this is not feasible. If only orientations are known, with an idea of the spacing or frequency of the discontinuities, then an assessment can be made of the probability or frequency of potential rock falls. On this basis, a rational determination can be made of the need for ground reinforcement (e.g., in the form of systematic or spot rock bolts or dowels) and the most effective orientation of such ground support.

(g) When tunnels are excavated by blasting, excess blasting energy at the perimeter will cause damage to the surrounding rock. This damage manifests itself as a loosening and weakening of the rock mass. With poor, uncontrolled blasting practices, the zone of damage can reach a distance of one to several meters. Joints and other planes of weakness may open temporarily or permanently due to the pressure of escaping gases or the dynamic, mechanical effect of the blast, thus eliminating any tensile strength that might have been available and reducing the shear strength. The blast will also create new fractures. Combined with the stress reduction due to the excavation of the opening, these effects greatly increase the opportunity for rock falls. An opening that would otherwise have been stable could require considerable ground support due to effects of poor blasting.

(h) Jointed and otherwise flawed rocks can be classified in many ways. One method of classification is described in Section 4-4, Terzaghi's classification of rock conditions for tunneling purposes. Additional comments are presented below.

(i) For purposes of underground design, intact rock may be described as rock in which discontinuities are spaced such that, on the average only about three to five discontinuities intersect the tunnel. Examples are massive igneous rocks, marbles, or quartzites with widely spaced joints, and sedimentary rocks that have been left largely unaffected by tectonics, dolomites, limestones, shales, and sandstones sometimes qualify.

(2) *Stratified rocks*. Stratified rocks are sedimentary or metamorphosed rocks with distinctive layering, where bedding planes are potential planes of weakness. Schistose rocks are typically metamorphosed rocks with layers or planes of weakness that are often greatly contorted.

(3) *Moderately and highly jointed rocks*. These rocks display few, if any, bedding plane weaknesses, but joints crossing the tunnel may number 10 to 100. Joints often come in patterns, with one to three sets of joints, each set containing mostly subparallel joints but the joint sets intersecting each other at angles. These kinds of rock are often called blocky or very blocky.

(4) *Interlocking rocks*. Interlocking, jointed rock masses can be moderately or highly jointed, but the joints are tight and contorted such that their inherent shear strength is high. Examples are some basalts, welded tuffs and rhyolites, and other rock masses where the jointing is largely the result of tension fracture from cooling soon after original deposition. Interlocking, jointed rock is often stable with a minimum of ground support.

(5) *Blocky and seamy rocks*. Blocky and seamy rocks combine jointing with weak bedding planes or schistosity. In sedimentary rocks, one or more joint sets are often seen at roughly right angles to the bedding planes.

(6) *Shattered or crushed rock*. This consists of mostly chemically intact fragments of rock, which may or may not be interlocking; the fractures are sometimes partly rehealed. Fault zones often contain rock that has been completely sheared into a silty or clayey material of low-strength, fault gouge. Such gouge is often responsible for squeezing conditions. The Karawanken case history (see Box 6-1) is a dramatic example of tunnel collapse in a fault zone. Missing in these descriptions is an indication of the degree of alteration and weathering. As earlier noted, weathering can have a profound effect not only on the strength of the joints but also on the intact rock strength. Recommendations for ground support based on these descriptions, intended for the design of steel sets, were formulated originally by Terzaghi. These recommendations are found in Chapter 7.

(7) *Rock failures affected by stresses*.

(a) Before excavation of an underground opening, the stresses in the rock mass are in a state of equilibrium. Excavation will reduce or eliminate the stress normal to the wall of the opening, while at the same time increase the stresses in the tangential direction through stress concentration, an effect similar to the development of stress concentrations around holes in plates. The effect of the increase in tangential stress depends on the strength of the rock, its ductility, and the stress distribution in the surrounding rock.

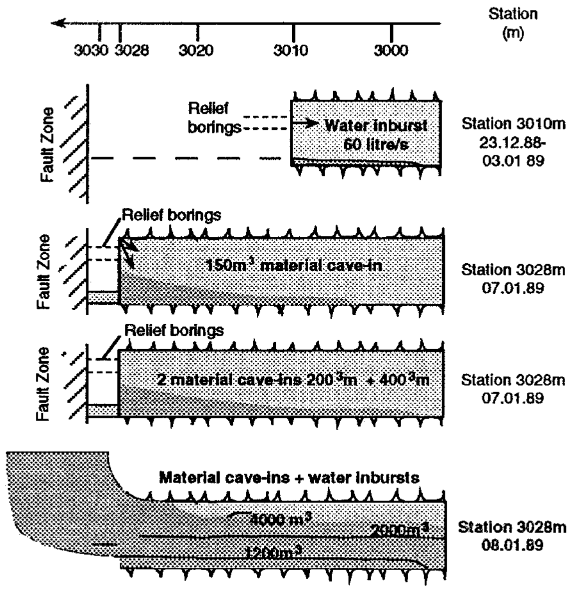
(b) If the rock is overstressed, it will yield or fail. A plastic, ductile rock (e.g., shale), behaving similar to a clay, may yield without losing coherence while the yield zone sheds load to deeper, unyielded rock. A fractured

Box 6-1. Case History: Karawanken Tunnel Collapse

The Karawanken Motorway Tunnel between Austria and Slovenia was built in 1987-91. The tunnel is 7.6 km long, with a 90-m^2 cross section and a maximum cover of nearly 1 km. This tunnel experienced a very large collapse during construction in 1988.

The tunnel traverses a variety of sedimentary rocks, ranging from dolomites and limestones to marls, clay shales, and conglomerates. The strata are severely folded and cut by a number of fault zones.

Excavation was by blasting methods, pulling 0.8-3.5 m with each blast, with a crown heading followed by bench removal at 80-150 m from the crown face. In some poor areas, two benches were employed. Ground support consisted of shotcrete varying in thickness from 50 to 250 mm, supplemented with rock bolts and steel mats as well as steel arches, based on a rock classification system. Where squeezing ground was encountered, open slots were left in the shotcrete application at the crown to permit displacements and rock relaxation. The construction procedure relied upon stabilization by pre-drainage, using horizontal bore holes from the face of the tunnel.



The collapse occurred at Sta. 3028, close by the Slovenia-Austria border and near the greatest amount of cover. Here is an abbreviated version of the series of events (see figure).

1. Dec. 23, 1988: At Sta. 3010, two exploratory borings encounter water, and large quantities of water and sand are released.
2. Dec. 27, 28: Five relief holes carry 60 l/s of water but soon collapse.
3. Jan. 3, 1989: Recommence driving after break; 1.2 m advance per round, relief drainage.
4. Jan. 7: At Sta. 3028 a crown borehole releases a water inburst, carrying 150 m^3 of material. Later a 500-mm drainage hole is drilled. This hole caves and delivers 200 m^3 of material, followed later by an additional 400 m^3 of material.
5. Jan. 8: The caved 500-mm hole is reopened by a small explosives charge. This is followed by more water and material inburst. Later on the same day the face collapses suddenly, releasing about 4,000 m^3 of water and material.

The causes of the failure were diagnosed to be a combination of at least the following factors:

1. Wide fault zone consists of crushed dolomite with sand and clay joint infill.
2. Removal of sand and clay material from the joint fillings result in loosening of the rock mass and loss of confining pressure.
3. Strength of the rock mass is reduced due to water softening, high water pressures (up to 35 bar), and reduced confining pressure.
4. Supporting pressure at the face was removed by excavation.
5. A contributing factor was the lengthy New Years break, during which water and fines were permitted to drain from the face.

Remedial measures consisted of placing a concrete bulkhead in the tunnel, constructing a bypass, placing a 5-m-thick ring of grout by injection, and careful remining.

If the potential seriousness had been recognized in time, the failure might have been prevented by grout injection into the entire width of the fault zone to make the zone impermeable and stable.

Reference: Maidl and Handke (1993).

rock, held in place by a nominal support of dowels or shotcrete, may yield with small displacements along fractures, perhaps with some fresh fractures, again shedding load to more distant, stronger rock. On the other hand, if the fractured rock is not held by a nominal force, pieces may tend to loosen, resulting in a stress-controlled raveling situation. A stronger, brittle rock will fracture and spall. A very strong rock can store up a great deal of elastic energy before it breaks, resulting, then, in occasionally violent rock bursts.

(c) The strength of intact rock as well as that of a fractured rock mass usually depends on the confining pressure. Just like a frictional soil material, the strength increases with the confining pressure or the minimum principal stress. Around an opening, the minimum principal stress is the pressure in the radial direction. Zero at the wall of an unlined opening, it increases rapidly when the wall curves but not when it is straight; the sharper the curve, the more rapid the increase in confining pressure.

(d) As it turns out, the highest stress concentrations are usually at the sharpest curves, such as the lower corners of a horseshoe-shaped opening, but here the confining pressure increases so rapidly with distance that a little local yielding tends to stop the process of failure. On the other hand, low-stress concentrations are often found around flat surfaces, such as flat roofs or floors (inverts). Here the stress gradients are small and stress fracture, when it occurs, can be very extensive. This is exacerbated in a rock formation that is horizontally stratified with little bond between the strata; here such stress conditions can lead to buckling.

(e) On occasion, tangential stresses induced over the crown of a tunnel will help confine blocks of rock that might have loosened in the absence of such a confining stress.

(f) Stress effects, then, depend on (at least) the following factors:

- Induced stresses, which depend on in situ stresses and opening shape, and the distance from the advancing face of the excavation.

- Rock strength; the intact rock strength can be measured; the operating parameter is the ratio between induced stress and rock strength, or if the induced stress is undetermined, the in situ overburden stress to rock strength ratio.

- Rock modulus and ductility, also measurable.

- Stress gradient, which can be calculated just as the induced stresses.

- Effect of fractures on strength and ductility, not measurable and barely possible to guess.

- Effects of stratification.

(g) Box 6-2 shows a method of assessing modes of failure based on induced stress level, rock strength, and rock quality. Box 6-3 describes various manifestations of stress-induced failure based on rock type and rock strength.

(h) As discussed later in this section, one type of stress-controlled failure is squeezing. This is a slow or rapid encroachment of rock material into the tunnel, without change in water content. In a soil, this would be likened to the squeezing or flow of a soft clay into the face of a shield, when the overburden pressure exceeds about six times the undrained shear strength of the clay. In a rock tunnel, squeezing conditions are often found in fault zones with altered or weathered material of low strength. At great depth where the stresses are high, a low-strength fault-zone material can result in a great deal of squeeze, and loads on a lining can approach the overburden pressure.

(8) *Failure modes affected by mineralogy*.

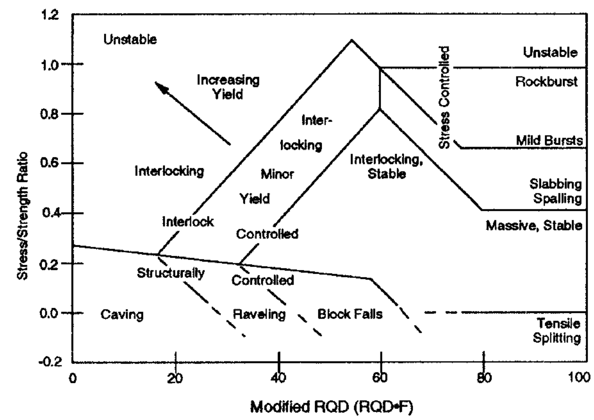
(a) Some modes of failure in tunnels are largely controlled by properties of the intact material. The concept that the strength of a massive rock affects stress-controlled failures such as spalling, rock bursts, or squeeze has already been discussed. Properties other than the rock strength also can result in failure or unacceptable behavior.

(b) Poorly consolidated shales or marls or shaley and marly layers in a limestone can slake when exposed to air and moisture. This is a phenomenon brought about by the stress relief combined with drying and wetting, and it appears in the tunnel as loosening of flakes or chunks of material, sometimes partly controlled by bedding. As pieces of the rock fall off, more rock gets exposed; slaking with time can result in the loosening and removal of several feet of rock. Slaking is greatly accelerated if water is permitted to enter the latent fractures of the rock and soften the rock. The risk of slaking can be assessed by means of laboratory tests, as discussed in Section 4-4.

(c) Saturated clay-like materials, when unloaded, will often generate negative porewater pressures (suction).

Box 6-2. Assessing Mechnaical Modes of Failure

1. Behavior of Strong and Brittle Rock Based on RQD and Induced Stresses



The following method of assessment was developed for nuclear waste repository design (Schmidt 1988) and is applicable to brittle, jointed, interlocking rocks, such as basalt, welded tuff or rhyolite, as well as other massive or jointed rocks, such as quartzite, marble, and most igneous and metamorphic rocks.

The method is based on the premise that massive rocks subjected to high stresses will suffer stress failure, but that flaws in the rock mass will permit relaxation of high stresses, leading to the potential for other modes of failure. The method requires the calculation of the stress/strength ratio, defined as the ratio between maximum tangential stress induced around an opening (calculated by closed solutions or numerical methods) and the unconfined compressive strength of the intact rock. The effect of flaws is assessed using a modified RQD, as follows:

$$\text{Modified RQD} = \text{RQD}\ F_1F_2F_3F_4F_5,$$

where

F_1 = factor for joint expression on a large scale (waviness), on a small scale (roughness), and continuity. Range 0.9 to 1.0 (1.0 for very wavy, rough, and discontinuous joints)

F_2 = factor for joint aperture and infilling, and joint wall quality. Range 0.92-1.0 (0.92 for soft or weakened joints)

F_3 = factor for joint orientation, favorable, random, unfavorable. Range 0.9-1.0

F_4 = factor for blast damage. Range 0.8-1.0 (1.0 for TBM tunnel, 0.8 for poor blasting)

F_5 = scale factor, function of ratio between opening size and joint spacing. Range 0.85-1.0

Ratio: Opening span/Joint Spacing	<4	4-10	10-30	>30
Factor F_5	1.0	0.96	0.88	0.85

The figure shows the predicted types of ground behavior based on stress/strength ratio and modified RQD. As most such charts, it is conceptually accurate, but the bounds between regions of behavior are imprecise and subject to judgment. For example, a jointed rock mass with joint blocks that are not interlocking (most tectonic joints) would most likely display a larger region of structurally controlled behavior.

These materials will absorb water either from the air in the tunnel or from distant regions in the clay mass, resulting in swelling. If unsupported, the clay mass will encroach on the tunnel profile; if lined, the tendency to swell will be halted but will result in lining pressures. Tertiary clays in Europe have been known to produce lining pressures greater than the overburden pressure. This is possible because these types of overconsolidated clays are usually subjected to in situ horizontal stresses greater than the vertical stresses.

(d) Prediction of swelling pressures in saturated clay or clay-shale has often been attempted using swell tests of the types used to predict swelling of unsaturated clays at the ground surface. However, because the swelling of a saturated clay in the underground is an entirely different phenomenon than the swelling of an unsaturated clay at the surface, these tests are useless for the purpose. Such underground swelling pressures, in theory, can be predicted by soil-structure analysis, but the necessary data to perform these analyses are difficult to obtain. Experience shows that the amount of swell of a clay or clay shale depends on the degree of cementation between clay particles; however, hard and fast general rules have not yet been established.

(e) Unsaturated clays or clay-shales are sometimes found in tunnels. These can be more prone to swelling than are the saturated materials, and standard swell tests performed on unsaturated samples can be useful. When

Box 6-3. Assessing Mechanical Modes of Failure

2. Manifestations of Stress-Controlled Failure

Unconfined Compressive Strength ksi	MPa	Typical Rock Types	Overstressed Behavior For Massive Rock	For Jointed Rock
64	440	dense basalt, quartzite, diabase, gabbro	violent regional, local rock bursts	
32	220	granite, most igneous rocks, gneiss, strong metamorphic marble, slate	breakouts in boreholes lesser rock bursts spalling, popping	combined failures (joints, intact rock)
16	110	hard, dense, sedimentary, welded tuff, dolomite, limestone	spitting, hour-glass pillars	
8	55	schistose rocks		
			flaking	
4	28	phyllite		
2	14	lower density sedimentary, chalk	stress slabbing	
1	7	tuff	slow slabbing	
0.5	3.4	marl, shale	squeezing slaking of poorly cemented shales	swelling accelerated by water access to joints
0.25	1.7	weak clay shale	swelling when cementation destroyed	
0.13	0.8	weathered and altered rock	ravelling of fissured clays	
0.06	0.4	hard clay	yielding of nonfissured clays	

Other effects:
· Stress-induced creep in halite, potash
· Swelling of anhydrite (up to 2 Mpa swell pressure with access to water)
· Dissolution of soluble materials

Note: Approximate lower limit for violent rock bursts: 18-24 ksi (125-165 MPa)

such materials are exposed to water during tunneling or due to leakage from the tunnel after completion, they can generate substantial swelling pressures. Such modes of behavior are accelerated by preexisting fractures (common in such materials) or fractures resulting from excavation and stress redistribution. The Peace River diversion tunnel case history (see Box 6-4) is an illustration of the effect of water on a silty shale.

(f) A common failure in weak, shaley rock, particularly in tunnels with a flat floor (horseshoe-shape) and high in situ horizontal stresses, is excessive floor heave. This type of failure is the result of several factors:

- For most in situ stress conditions, a flat floor results in very low vertical and often high

Box 6-4. Case History: Diversion Tunnel in Soft Shale, Peace River

For the Site 3 hydroelectric project in British Columbia, three diversion tunnels through the left abutment were proposed. Confidence in the behavior of the soft shale was not great, and a test chamber in the shape of a truncated cylinder, 11.1 m wide, 7.5 m high, and 45 m long was excavated. The chamber is at a 107-m depth and connected to the canyon wall through an adit. Most excavation was by roadheader, but part of the adit and part of the chamber were excavated by controlled blasting.

The geologic material is a Cretaceous, horizontally bedded silty shale with about 10 percent smectite, with unconfined compression strength 6 Mpa (900 psi) and modulus 3-4 GPa (440-580 ksi) perpendicular to bedding, 6-8 Gpa (870-1,160 ksi) parallel to bedding. The material is prone to slaking and weathering when exposed. Bedding plane fractures are common, as are steeply dipping relaxation joints parallel to the canyon wall.

Ground support included two layers of fiber shotcrete and tensioned resin dowels spaced 2 m. The chamber was instrumented with convergence gages, multipoint extensometers, and stress cells.

The chamber was successfully excavated and supported, using heading and bench. Shotcrete in the roadheader section was generally sound, with minor shrinkage cracking, but in the blasted section up to 65 percent of the shotcrete was drummy.

After completion, the chamber filled with water, 5.5 m deep, for about 2 years; it was then pumped dry and inspected. Shotcrete in the crown, which remained dry, had remained virtually unchanged and sound. Below the water line, the shotcrete was badly cracked and spalled, and drummy throughout. Two block falls of 100-150 m^3 each had occurred, bounded by clean joints parallel to the tunnel wall. Cores were taken, and shale from the wet zone was found to be soft and fissile. Ground movements in the dry crown were about 0.3 mm, but in the wet zone, ground movements amounted to 50-120 mm.

Conclusion: Shotcrete-shale bond was a problem if the shotcrete was not applied quickly; more so in the blasted than the mechanically excavated parts. Water found its way through cracks and voids in the shotcrete into existing and latent fissures in the shale, where it caused softening and swelling, and resulted in displacement and spalling of shotcrete. The diversion tunnels are to be designed with a circular shape and a cast-in-place concrete lining over the initial shotcrete support.

Reference: Little (1989)

horizontal stresses in the floor, conducive to swelling of the floor material.

- Seepage water finds its way to the floor, causing swelling.

- The floor is subject to construction traffic, which causes softening in the presence of water.

Swelling also occurs when geologic materials such as anhydrite or shales containing anhydrite absorb water.

(9) *Effects of water*.

(a) As discussed earlier, groundwater contributes to modes of behavior such as swelling and slaking. Water can contribute to many other modes of behavior and failure.

(b) Some rocks or minerals are soluble in water. These include, most notably, halite (rock salt) and gypsum. Moving water will carry away salt and gypsum in solution and leave behind voids that can cause increased water flow and accelerating dissolution. Voids can cause surface subsidence or irregular loading and loss of support for tunnel lining and the ground support system. Removal of the gypsum cement in a sandstone by seepage water has caused the failure of at least one major dam (San Franciscito Dam in California, in 1928). When such materials are present, particular attention must be paid to the watertightness of the tunnel.

(c) In the longer term, limestone is also subject to dissolution. In this case, however, the concern is more for the likelihood of encountering voids and caverns than the prolonged effect of dissolution on the tunnel structure.

(d) Flowing water will erode unconsolidated material. Piping phenomena are common in soils, where backward erosion by seepage water can cause failure of dams and excavations as well as cut slopes. In rock masses, joint fillings and crushed fine materials in faults and shear zones are particularly susceptible. In a construction situation, prolonged water flow out of joints and shear zones can cause serious weakening of the rock mass by removal of fines, resulting in loosening and potentially collapse (see

Box 6-1 on the Karawanken Case History). Contributing factors in such situations are the weakening effect of the water on the strength of intact rock and joints, joint fillings, and gouges; the hydrostatic pressure reducing the effective stress across joint surfaces; and the seepage forces of the flowing water.

(e) Inflow into tunnels loaded with silt and sand will cause maintenance problems for dewatering pumps. An open TBM is not greatly affected by water inflow, but a shielded TBM often suffers problems when inflows exceed several tens of liters/second (several hundred gpm), especially when the water brings in fines. Often the mucking system, whether by rail cars or conveyor, is overloaded by the water, and water with fines escapes the system, resulting in deposition of fines at locations where they will be troublesome. As an example, silt deposited in a telescoping shield joint will cause wear in the joint and may destroy waterproofing gaskets. Silt deposited in the invert can seriously hamper placement of invert segments. Excess water can also affect the electrical system and cause corrosion of tunneling machinery, especially if the water is saline or otherwise corrosive.

(f) Inflow into tunnels will tend to drain the rock mass and any overburden. This, in itself, may be unacceptable, especially if existing flora or operating wells are dependent on maintenance of the groundwater table. Lowering the groundwater table can also result in consolidation of unconsolidated materials, especially soft clays, resulting in unacceptable surface settlement.

(g) A particular type of failure mode applies to water tunnels in which the water pressure fluctuates, such as in power tunnels with surges and water hammer effects. If the tunnel is unlined or supported only by rock bolts or dowels, the fluctuations in water pressure can result in water flushing in and out of rock fissures, eventually cleaning out joint fillings. This also happens if there are cracks in a tunnel concrete or shotcrete lining that permit the flushing of joints. More than one power water tunnel has failed by collapse in this way.

(10) *Particular failure modes for shotcrete*.

(a) Before reviewing failure modes for shotcrete ground support, it is useful to recapitulate the various functions and actions of shotcrete support, when applied to a minimum thickness of 50-75 mm:

- Sealing coat to prevent atmospheric deterioration, slaking, drying, wetting, swelling.
- Prevents blocks of rock from falling out by shear and bond strength; prevents smaller fragments from falling and start a raveling sequence.
- By shear, bond, and bending to withstand local forces or forces of limited extent (local blocks, seams subject to squeezing or swelling).
- As a compression arch or ring, to withstand more-or-less uniform loading from squeezing, swelling, or creeping ground.
- Provide some degree of water inflow control.
- In combination with rock bolts or dowels, provide overall stabilization and ground movement control.

(b) Overall, by inhibiting ground motions and supplying a confining pressure for the rock mass, the shotcrete acts to retain and improve the strength of the rock mass and to help in creating a self-supporting ground arch in the rock mass.

(c) Where shotcrete is a part of initial ground support, to be followed by subsequent installation of a final lining (whether by cast-in-place concrete or additional shotcrete), performance requirements are less stringent than when shotcrete is the final support. Shotcrete as initial ground support can be repaired and even replaced as required, and even significant flaws can be tolerated, provided they do not impair the safety of personnel. The principle of controlled deformation of initial shotcrete support is discussed further in the section on the New Austrian Tunneling Method (Section 5-5).

(d) Some failure modes of shotcrete result from imperfections in its application, others from properties and nonuniformities of the rock mass, or the action of formation water. Some examples follow:

- Shear failure resulting from loss of (or lack of) bond between rock and shotcrete, usually initiated by nonuniform loading combined with an incomplete ring of shotcrete.
- Shear failure from local block load or load from a seam of squeezing material.
- Compression failure from excessive external uniform or nonuniform load, sometimes a combined bending and compression failure.

- Fracture due to excess external water pressure, resulting in excessive water inflow, sometimes resulting from plugging of geofabric strips and piping provided for draining the rock mass.

- Shear failure of shotcrete around a rock bolt or dowel plate resulting from excessive displacement (squeeze) of the rock mass.

(e) Loss of rock-shotcrete bond can result from incomplete preparation of a wet, partly deteriorated rock surface or one covered with grime, dust, or mud. Other common flaws are areas with too little or too much aggregate, too high water/cement ratio, imperfect application of admixtures resulting in slow curing, or too thin an application. Application of shotcrete in a location with flowing water can result in washouts or imperfect bonding or curing.

(f) The case history in Box 6-4 shows failure modes of shotcrete exacerbated by fractures in the shotcrete and softening of the rock.

(g) Many potential modes of failure of a shotcrete application are functions of flaws in shotcrete application and local variations in geology and loading, generally not subject to analysis but usually controllable during application. Where the shotcrete forms a structural arch or ring bonded to the surrounding medium and subject to external loads, the shotcrete structure is amenable to analysis.

(11) *Failure modes of rock bolt or dowel installations*.

(a) Rock bolts or dowels can control or reduce displacements, both initially and in the long term, by preventing loosening of the rock mass and increasing the rock mass modulus to hold rock blocks or wedges in place. In a pattern, they act to form a reinforced arch or beam capable of sustaining loads that may be uniform or nonuniform. By preventing loosening of the rock mass and by increasing the rock mass modulus, bolts and dowels control or reduce displacement in the short or long term. Prestressed bolts induce compression in the rock mass, further increasing its strength and carrying capacity and reducing displacements. Bolts and dowels are often supplemented by metal straps, wire fabric, or shotcrete.

(b) Bolt or dowel installations may be considered permanent parts of the underground structure, or they may be temporary and not counted on for permanent support. The installation may be supplemented at any time with additional ground support elements.

(c) Individual dowels or bolts can fail in either shear or tension in the steel, or yield can occur along the bond between grout and rock, or between metal and grout. Sometimes failure occurs due to faulty installation (insufficient grout, grout not properly set, improper anchoring).

(d) A systematic bolt or dowel installation can fail by loosening, raveling, or block fall between individual bolts; this depends on joint spacing relative to bolt spacing and the degree of interlock between rock blocks. If bolts are too short to anchor a large wedge, such a wedge can fall out, bringing down one or several bolts with it.

(e) A systematic bolt or dowel installation forming an arch or a beam can fail due to overstress of the reinforced rock mass. This usually indicates that the bolt length chosen was too short.

(f) In a soft, squeezing ground, bolt face plates can fail by overload in the metal or by punching failure into the rock.

(g) Whether any of these modes of performance have serious consequences depends on the permanency of their installation. Systems installed for temporary purposes only are considered to perform acceptably as long as there is no hazard to personnel and the permanent lining can be installed without problem. The temporary installation is employed to arrest ground movements before permanent lining installation.

(h) When the bolt installation is considered as part of the permanent installation, some of these modes of failure may still be acceptable. Yielding of part of the system (shear, tension, bond) may be acceptable as long as the rock mass is coherent and deformations are under control. However, their value may have to be discounted for the design of the final lining. Any behavior mode that can result in future corrosion, however, usually requires that the element is ignored for final design consideration.

(12) *Particular failure modes for shafts*.

(a) Because shafts are oriented 90° from tunnels, some modes of failure are more or less common than for tunnels. There are several reasons for that. First, since a shaft penetrates the geologic strata in a vertical direction, a shaft is likely to encounter a greater variety of conditions, including overburden and weathered rock. Second, gravity acts on the shaft wall like on a tunnel wall, much less severely than on the crown of a tunnel. Third, methods of shaft construction are generally very different from methods of tunnel construction, as discussed in Section 5-7. The following are a few examples of shaft failure mechanisms.

(b) Shaft bottom failure is usually caused by water pressures. With an impervious plug above an aquifer at the bottom of the shaft, the plug can fracture or burst if it is too thin and cannot hold the pressure, whether by bending failure or shear along the sides, or some combination. Of course, sinking the shaft and ignoring the aquifer altogether could result in flooding of the shaft, if the permeability in the aquifer is sufficiently great.

(c) Grouting or freezing is often used to control groundwater inflow and the effect of groundwater pressures during shaft construction. It is difficult to ascertain the quality of grouting, and ungrouted zones can be left that would result in excess inflow of water, perhaps carrying solids, when encountered during sinking. A freeze-wall occasionally fails, also resulting in inrush of water, often because flowing groundwater brings caloric energy to the site and thaws the wall.

(d) Another shaft failure mode has nothing to do with rocks or groundwater but with the site arrangement: flooding of the shaft from surface waters. This type of incident is inexcusable; shafts constructed anywhere near a floodplain must be equipped with a collar tall enough to prevent flooding.

(13) *Particular failure modes at portals*. Portals are typically cut into the hillside and preferably expose sound rock. The portal cut is exposed to all of the failure modes of any man-made cut into soil, colluvium, talus, or rock, including slope failure on a discontinuity plane, rock falls, deterioration due to exposure, deep-seated failures, sliding of overburden materials on top of bedrock, etc. Fractures are often opened in the ground due to the excavation, and if filled with rain water, the water pressure can result in failure initiation. Rockfalls can be hazardous to personnel moving in and out of the tunnel. In addition to the typical slope failure phenomena, the portal is also the intersection between the tunnel and the portal cut. Tunnel excavation by blasting, if not carefully controlled, can result in very large overbreaks. For these reasons, the ground surrounding the tunnel must be carefully supported, and the initial tunnel blasting performed with low energy, as discussed in Chapter 5.

b. Failure modes of tunnels and shafts during operation. Most of the modes of failure discussed above apply to the construction environment; once they are dealt with, they pose no further threat. Some of the conditions responsible for the failure modes, however, can also affect long-term performance, especially if they are not dealt with properly. Following are additional modes that apply, typically, to the finished, lined structures.

(1) *Failures due to water pressure*.

(a) Internal water pressure can result in fracture of a concrete lining and escape of the water into the formation. If these formation water pressures cannot dissipate (as in a permeable formation), the formation may be fractured by hydraulic jacking, with the potential for tunnel damage, or worse—instability of adjacent slopes or valley walls. This phenomenon is discussed in Chapter 9. Such failures can occur if the lining is not designed for the hoop tension caused by the internal water pressure and the formation (and formation water) pressure on the exterior is lower than the internal pressure.

(b) The principal failure mode of concern for external water pressure is the buckling of steel-lined tunnels. During operation a steel-lined pressure tunnel is not in danger due to external water pressure, but the empty tunnel must accept the full external pressure without internal balancing pressure. Not infrequently, leakage from the pressure tunnel causes the formation pressure to rise to a value close to that in the tunnel. When the tunnel is then emptied, it has to withstand an external pressure equivalent to the internal pressure.

(c) A tunnel lining is often furnished with an impervious membrane to control groundwater inflow that would otherwise be excessive. As a general rule, this impervious membrane must accept the full external water pressure and be supported by an internal structure capable of withstanding this pressure.

(2) *Tunnel lining failure caused by external loads*.

(a) The failure of a concrete tunnel lining has to be viewed in terms of its functional requirements. A tunnel lining may crack or leak or deteriorate, but as long as it serves its function for the expected lifetime, it has not failed.

(b) The following discussion, for the most part, applies equally to cast-in-place and precast, segmental lining. Tunnel linings in rock are externally contained; they are different from aboveground structures for at least the following reasons:

- Stresses and strains are governed not so much by loads as by interaction between the lining structure and the ground requiring compatible displacements.

- Except for water pressure, loads on the lining often relax upon displacement and yield; they are not conservative or following loads.

- Radial fractures in a concrete lining do not usually form a mechanism of instability, witness voussoir arches without bonds between blocks. The compressive stress between adjacent blocks combined with friction between the blocks suffices to maintain the stability of the arch, even with a substantial external load.

- Because of net hoop compression (in a circular tunnel, often also for other shapes), a tension fracture from the inside face due to bending does not usually penetrate the thickness of the lining.

- The rock surrounding a tunnel lining is usually under relatively strong compression, and the bond between lining and rock is usually good. Therefore, tendencies to generate external tension fractures due to bending are greatly resisted.

- The usual circular shape is inherently strong and forgiving and, with usual dimensions, resists buckling. Horseshoe and other shapes are not as forgiving.

(c) Structural failure of a concrete lining does occur on occasion. When it does it is usually for one of the following reasons:

- Loss of support around part of the lining due to inadequate concrete placement or contact grouting, especially in the crown of the tunnel, or due to washout of fines, dissolution, or rotting of timber, resulting in uneven loading and support. Unrelieved differential hydrostatic pressures can also exist in such void spaces during filling or emptying of the tunnel.

- Excessive or nonuniform load on a circular lining, causing large distortions, sufficient to create compressive failure in bending (rarely by uniform thrust); nonuniform load may be caused by stratigraphic or structural geologic differences across the tunnel section and by nonuniform swelling or squeezing.

- Excess side pressure on walls of horseshoe-shaped tunnel, resulting in gross bending of the walls or buckling of the floor, or both. This can also result from loss of floor strut due to excessive floor heave.

- External factors, such as effects of adjacent new construction, slope failure at a portal or in the vicinity of the tunnel.

6-4. Seismic Effects on Tunnels, Shafts, and Portals

It is generally acknowledged that underground structures are inherently less sensitive to seismic effects than surface structures. The good performance of underground structures was demonstrated during the 1986 Mexico City earthquake, where subway structures in soft and very soft ground went undamaged and the subway served as the principal lifeline, once power was restored. In contrast, buildings and other surface facilities suffered severe damage. Nonetheless, underground structures can suffer damage in an earthquake under particularly unfavorable conditions. In most cases, however, the vulnerability of a particular structure can be assessed and a design prepared that will eliminate or minimize the effects of earthquakes. The vulnerability of underground structures is examined in Box 6-5.

a. Effect of earthquake shaking on tunnels and shafts.

(1) Earthquake waves traveling through the ground are displacement waves, generally compression (P) or shear (S) waves. Due to scattering and other effects, the seismic displacement waves can vary nearly randomly in space and time. The response of a tunnel or shaft is either axial compression or extension, horizontal or vertical curvature, or ovalizing (racking), or usually a combination of all.

(2) A tunnel or shaft structure subjected to axial and curvature motions may be compared with a beam under combined compression (extension) and bending-maximum and minimum stresses occur at the extremities. The resulting stresses depend on the initial static stresses, upon which the dynamic motion is superimposed.

(3) Ovaling may occur due to a shear wave impinging nearly at a right angle to the tunnel or shaft. While one diameter is increased, the perpendicular diameter is reduced a similar amount, and moments are created

Box 6-5. Case History: Tang-Shan Eearthquake, 1976; Performance of Underground Structures.

The Tang-Shan Earthquake was of Magnitude 7.8, with surface Mercalli intensities of X to XI. It occurred in an industrial area with several coal mines. Surface faulting extended for more than 10 km, and fault traces with displacements up to 1.5 m traversed underground mine facilities. On the surface, destruction was nearly 90-percent complete, and several hundred thousand lives were lost. Damage to underground structures, however, was relatively minor, and all miners, some 1,000 in number, were evacuated safely.

An incline provides access to the Tang-Shan mine, located in the area of greatest surface destruction. The inclined tunnel passes through 4 m of clay and a 62-m strata of limestone before reaching shale and coal strata. The tunnel is horseshoe shaped (arch and straight walls) and lined with bricks or stone blocks, with an unreinforced concrete floor. The tunnel is 1.8-2.5 m high and 1.2-2.5 m wide. Tunnel enlargements for electrical and pumping gear are 2-3 m high and 3-5 m wide. These structures remained essentially intact and passable after the seismic event.

The first 15 m of tunnel through the clay experienced circumferential cracks 1-3 m apart and 10-50 mm wide; a horizontal crack, 20 mm wide, also occurred. Down to a vertical depth of 30 m, the spacing of cracks decreased to more than 10 m, with up to a 10-mm crack width. Beyond this depth, there were occasional cracks. The concrete floor of the pump station at a 30-m depth heaved up to 300 mm and experienced a crack 10 m long, and a few bricks and pieces of plaster loosened and fell. The station at a 230-m depth experienced a floor heave of 200 mm along a length of about 7 m. The station at a 450-m depth showed a 50-mm floor heave in a 1-m area; only small pieces of plaster fell off roof or walls. Damage was noted mostly at weak spots, such as at changes in cross section or lining material, or at bases of arches. There was clearly a great reduction in damage as a function of depth; but on the whole, the tunnel remained intact and passable.

In contrast, pumps and transformers in the underground were damaged; many transformers toppled over. Rail cars tipped on their wheels and lifted up to 30 deg off their rails. People in the mine corridors were thrown into the air up to more than 0.3 m or along horizontally several meters, indicating accelerations greater than one g.

Production drifts in the coal mines, designed and built for a limited lifetime through weak rocks, saw effects such as excessive loading of hydraulic mine struts, breaking of support timber, loosening and fallout of chunks of coal, dust filling the air, squirting of water out of fractures during the earthquake motion, and increased water flow through fractures in general. Most of this behavior occurred within a distance of some 100-150 m from the faults actually observed being displaced. Beyond this range, the mine openings, though violently shaken, showed little permanent damage.

This case history demonstrates the survivability of even poorly supported tunnels and other underground openings through relatively weak rock when subject to violent earthquake motions.

Reference: Wang (1985)

around the opening. Maximum and minimum stresses occur at four points around the opening, at the inside or outside surface of the lining, or tangential to the rock surface in an unlined tunnel.

(4) Regardless of the motion induced by an earthquake, the result is manifested as extension or compression at points around the tunnel or shaft opening. Tensile stresses can occur if the initial tangential stress (usually compression) is small. These transient stresses can usually be considered as pseudo-static superposition on the existing stresses, because the seismic wavelength is almost always much longer than the dimension of the typical underground structure. There is little dynamic amplification, because the resonant frequency of an underground opening is much higher than the typical frequency band of seismic waves. Studies suggest that dynamic stress amplification at the tunnel opening generally gives stresses that can be up to 10 or 15 percent higher than pseudostatic solutions. This is different from typical surface structures (buildings, bridges), whose natural frequency often falls within the typical seismic wave frequency band, and where amplification can be large.

(5) In an unlined tunnel, shaped to have its circumference generally in compression, the additional seismic stresses are generally inconsequential. Blocks of rock that are almost ready to fall can loosen and fall out due to the shaking. Even when tension cracks occur, or existing cracks open, they will typically close again in a fraction of a second, without consequence. Similar arguments apply to a tunnel supported with spot bolts and occasional shotcrete support.

(6) Where a pattern of tensioned bolts has been applied as ground support, the bolts create a compression

ring around the tunnel or cavern arch, preventing tension and holding blocks in place. Similar conditions prevail with untensioned pattern dowels and shotcrete support, where ground motions have induced some tension in the dowels to form a compression arch.

(7) A concrete lining will be subject to compression and extension at points on the exterior and interior of the lining. As discussed in Chapter 9, exterior extension is of no consequence. In the event that tension cracks appear on the interior surface, they will close again after a fraction of a second. Such cracks do not usually extend through the thickness of the concrete and cannot, in themselves, form a failure mechanism. A simplified method of analyzing tunnels in rock for seismic effects is shown in Box 6-6. This simplified method ignores the effect of ground-structure interaction and provides an upper-bound estimate of strains induced in the lining. The method permits a quick verification of the adequacy of the lining design in reasonably competent ground. In very weak ground, ground-structure interaction should be considered to avoid overdesign of the lining.

b. Effects of fault displacement.

(1) Tunnel alignments should avoid active faults whenever possible; however, if faults cannot be avoided, the design must include fault displacement. It is not possible to build a structure that will resist the fault displacement. If the tunnel structure is to remain functional after the earthquake, strategies must be planned to mitigate the effects of fault displacement.

(2) For rail tunnels, the strategy has been to build the tunnel oversized through the fault zone, sufficient to realign the track with acceptable lateral and vertical curves after the event, while reinforcing the ground in and around the shear zone sufficient to prevent collapse. A ground reinforcement system of great ductility is required, such as a combination of lattice girders, wire mesh, rock dowels, and shotcrete. Tunnel damage is expected; however, repairs can be quickly accomplished.

(3) For shallow water tunnels, the most effective solution may be to plan for excavation and replacement of the damaged structure after the event. In a deeper tunnel, repair and replacement may not be so easy. In this case, the tunnel may be oversized through the fault zone and a relatively flexible pipe constructed within the tunnel, providing enough space to avoid shearing the pipe due to the fault motion. The pipe must be supported or suspended to permit motion in any direction.

c. Other permanent displacements of the ground. Portals are particularly vulnerable to permanent displacements during earthquake events. Slope stability in the event of an earthquake can be analyzed using dynamic slope stability analyses, and portal slopes can be reinforced, using tieback anchors or other devices as necessary. Another potential problem is falling rocks, loosened by the earthquake. Large blocks of rock loosening may be secured individually, or shotcrete may be applied to prevent loosening.

Box 6-6. Seismic Analysis of Circular Tunnel Linings (Continued)

1. Longitudinal Bending and Extension or Compression

Obtain seismic input parameter from seismologist:

V_s = maximum particle velocity from shear wave

A_s = maximum particle acceleration from shear wave

Obtain effective shear wave propagation velocity C_s of rock medium from in situ seismic survey or from relationship with effective shear modulus G (under earthquake shear strain level):

$C_s = \sqrt{}G/\rho$

where ρ = specific gravity of rock mass. Shear modulus is related to Young's modulus E_r by

$G = E_r/2(1 + v_r)$

where v_r is Poisson's ratio for the rock mass

With the assumption that the tunnel structure is flexible relative to the ground, then the tunnel structure will conform to the free-field motion of the ground, and the maximum and minimum (compression, extension) strain of the tunnel structure is

$E_{max/min} = \pm(V_s/C_s) \sin\theta \cos\theta \pm (A_sR/C_s^2) \cos^3\theta$,

where R = tunnel radius (strictly speaking, R = distance from extreme compression fiber to neutral axis) and θ = angle of incidence of seismic shear wave. The greatest/smallest strain is usually found for $\theta = 45°$:

$E_{max/min} = \pm 0.5\ V_s/C_s \pm 0.35\ A_sR/C_s^2$ -

2. Ovaling or Racking

A seismic shear wave impinging on a circular tunnel structure at a right angle will cause the structure to rack or ovalize, shortening one diameter D by Δ D and lengthening the orthogonal diameter by an equal amount. In the free field rock mass, the shear strain can be approximated by

$\gamma_{max} = V_s/C_s$,

and an unlined hole driven through the rock mass would suffer an ovalizing distortion of

$\Delta D / D = \pm \gamma_{max} (1 - v_r)$

The maximum strain in the lining, then, is

$E_{max} = V_s/C_s\ [(3(1 - v_r)t/R + 1/2\ R/t\ E_r/E_c\ \{(1 - v_c^2)/(1 + v_r)\}]$

where t = lining thickness, R = tunnel radius, E_c = concrete modulus, v_c = Poisson's ratio for concrete.

3. Notes

Ovalizing strains are superimposed on strains pre-existing from static loads.
For a maximum earthquake design, usable compressive strain is about 0.003.
Tension cracks due to excessive extension dynamic strains usually cannot be avoided. They will, however, generally close again after the seismic event. Tension cracks can be reduced in size and distributed by appropriate crack reinforcement.

Box 6-6. (Concluded)

4. Example - Los Angeles Metro, Circular Tunnel in San Fernando Formation

A_s = 0.6g, V_s = 3.2 ft/sec, C_s = 1360 ft/sec

R = 10 ft, t = 8.0 in., $E_c/(1 - v_c^2)$ = 662,400 ksf, E_r = 7200 ksf, v_r = 0.33

1. Longitudinal:

$E_{max/min}$ $= \pm 0.5 \times 3.2/1360 \pm 0.35 \times 0.6 \times 32.2 \times 10/1360^2$

$= \pm 0.00118 \pm 0.000037 = \pm 0.00122 < 0.003$ - ok

2. Ovalizing:

Δ D/D = + 2 * 3.2/1360 (1 - 0.33) = 0.0031

$E_{max/min}$ $= \pm 3.2/1360\ [3(1 - 0.33)(8/120) \pm 1/2 * 120/8 * 7200/ (1 + 0.33) \times 1/662{,}400)]$

$= \pm 3.2/1360\ (0.134 + 0.122) = 0.0006 < 0.003$ - ok

This example is for a concrete tunnel through a weak, soil-like material. Tunnels through stronger, rock-like materials would be subjected to lower seismic strains.

Reference: Wang (1985)