FCT Quality Assurance Program Document

Appendix E
FCT Document Cover Sheet

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☒ Technical Review ☐ Peer Review

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Review of Underground Construction Methods and Opening Stability for Repositories in Clay/Shale Media

Fuel Cycle Research & Development

Prepared for
U.S. Department of Energy
Used Fuel Disposition
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June, 2014
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Sandia National Laboratories

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Acronyms

BBM	Barcelona Basic Model
COX	Callovo-Oxfordian argillite (host rock at the Bure URL)
CSO	Combined-Sewer Overflow
DPC	Dual-Purpose Canister
ECT	Euclid Creek Tunnel
EDZ	Excavation Damage Zone
EPB	Earth Presssure Balance (TBM design)
ETH	Zurich Technical University
FHA	Federal Highway Administration
HLW	High-Level Waste
ILW/LLW	Intermediate- and Low-Level Waste
LLW	Low-Level Waste
R&D	Research & Development
RH	Relative Humidity
RQD	Rock Quality Designation
SNF	Spent Nuclear Fuel
SS	Slurry Shield (TBM design)
TARP	Tunnel and Reservoir Plan (Chicago)
TBM	Tunnel Boring Machine
THM	Thermal-Hydrological-Mechanical
UCS	Unconfined Compressive Strength
URL	Underground Research Laboratory

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Review of Underground Construction Methods and Opening Stability for Repositories in Clay/Shale Media

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June, 2014

1. Introduction

This report reviews the art and practice of excavating and constructing underground facilities in clay/shale media, as part of a multi-year evaluation of the technical feasibility of direct disposal of spent nuclear fuel (SNF) in dual-purpose canisters (DPCs). The purpose is to review worldwide examples of large-scale excavations in clay/shale media, the methods used for excavation and construction, and the costs. It is anticipated that this information will help to show the feasibility of construction for a deep geologic respository for (on the order of) 10,000 large, heavy, heat-generating waste packages. This report refines the clay/shale disposal concept for DPC-based waste packages, in support of future studies that include cost estimation.

Earlier studies compared reference concepts for disposal of SNF and high-level waste (HLW) in various media (clay/shale, crystalline and salt) with one result being that construction and maintenance costs for openings in clay/shale were considered to be relatively high (Hardin et al. 2012). A later study adopted in-drift emplacement instead of large-diameter emplacement boreholes, for large, heavy DPC-based waste packages. This change would increase by several-fold the length of emplacement drifts needed to dispose of all the commercial SNF projected to produced in the U.S. (Hardin et al. 2013). This in turn raised questions of engineering feasibility: whether ~300 km of emplacement drifts could be efficiently constructed, then remain stable for at least 50 years during which the waste packages would be ventilated, with little or no maintenance. This report affirmatively answers these questions, based on experience with similar tunnels that have been in service for highways, railroads, and water conveyance for durations approaching 50 years or longer, in the U.S. and Europe.

This work supports the evaluation of technical feasibility of direct disposal of DPC-based waste packages, in accordance with an existing work plan (Howard et al. 2012). One application will be to support cost estimates for DPC direct disposal to supplement previous documentation developed for reference disposal concepts (Hardin et al. 2012).

This topic is strongly related to direct disposal of DPCs (and less important for smaller, purpose-built canisters) because the DPC-based packages would be larger and hotter. Because of the size and weight, in-drift emplacement would be desirable. Because of DPC heat output and limitations on heat dissipation in clay/shale media, ventilation of approximately 300 km of repository emplacement tunnels for at least 50 years could also be needed. The topic was identified as an R&D need in a previous, preliminary report on feasibility evaluation (Hardin et al. 2013, Section 10).

Clay/Shale Terminology – The clay/shale terminology used in this report is intended to include a spectrum of rock types including plastic clay, claystone, mudstone, siltstone, argillites and shales. Hansen et al. (2010) provided a working definition that will be used here:

> "Sedimentary rocks are classified by the predominant grain size of their constituent materials and other textural parameters such as layering, and the composition or mineralogy of the constituent grains (e.g., clay minerals, carbonates, or quartz). Clay is a term used to describe rock-forming argillaceous minerals, rock fragments rich in clay minerals, or a detrital particle of any composition smaller than a very fine silt. Fine-grained constituents include sand, silt, and clay-sized grains (in order of descending grain size) and come from weathering of rocks. The sedimentary rocks composed predominantly of the finest sediments, clay and silt, are claystone and siltstone. If both clay and silt are present and the rock has a fine laminated or fissile texture, it is called shale. Unconsolidated silt and clay sediment together make mud, and shale is made of indurated mud with fissile lamination. Mud may be unconsolidated, such as the plastic Boom clay in Belgium, which is considered a potential repository host formation. Mudstone is a lightly indurated mud having the texture and composition of shale but lacking the fine lamination or fissility. Argillite is a compact rock derived either from mudstone, claystone, or siltstone that has undergone a somewhat higher degree of induration than mudstone or shale, but is less clearly laminated. Argillaceous rock is slightly different from argillite in that 'argillaceous' describes a rock formed predominantly from clay-sized or clay mineral particles." (Hansen et al. 2010, Section 1.2)

Clay/shale rock types considered for repository construction typically would be rich in clay minerals (mainly smectites and illites). Suitable clay/shale rock types may be collectively referred to in this report as argillaceous, specifically addressing those rich in clay minerals (the term can also be used for very fine grain detrital rocks with lower clay mineral content). Hansen et al. (2010) described the ideal clay/shale host medium as fine grained, lightly indurated (too much induration will eliminate plasticity, e.g., as in slate), detrital sediment (i.e., weathering products deposited in water) with approximately 50% or greater clay mineral content and low permeability (on the order of 10^{-19} to 10^{-16} m^2).

Repository Depth – The typical depth for a repository in clay/shale media would be 300 to 900 m (Shurr 1977). A shallower repository might be considered to accommodate unit thickness and depth, if the desired characteristics of geologic stability and waste isolation can be achieved. For example, the plastic Boom Clay is a candidate host medium for construction of a repository at a depth of approximately 250 m (ONDRAF/NIRAS 2001). Were it to occur at much greater burial depths, this clay could be less porous and more indurated (i.e., diagenetically modified). A plastic clay subject to loading from hundreds of meters of overburden could require complicated excavation and ground support measures. The feasibility of repository construction and long-term operation in clay/shale media is strongly related to the burial depth, the associated stress and environmental conditions in the rock around a repository, and the rock response to those conditions.

Groundwater Effects – Clay/shale media are hydrologically saturated, in that all interstices (macropores, nanopores and inter-layer spaces) are fully occupied by water. Clay minerals are water sensitive, and loss of water during construction and operations can be destabilizing. At the same time, clay/shale media with low permeability may be very slow to produce groundwater into excavated openings, and much of the water in the undisturbed material apparently does not behave as mobile, free water. There currently is scientific debate about the importance of coupling between stress and pore pressure, at least in some clay/shale media (Pellet 2014, verbal communication). What this means for excavation and ground support is that water inflow and hydraulic loading are secondary concerns, at least for time scales on the order of years to decades. Whereas water inflow is observed in a few boreholes at the Bure underground research laboratory (URL) and the Mont Terri URL, these are isolated occurrences representing channelized flow that can have only local impact on repository construction and operation.

Other Generic Host Media – The feasibility of repository excavation and ground support in other generic host media, particularly salt and crystalline rock, is less uncertain than in soft sediments. Crystalline rock that would be considered for a repository is much stronger and less plastic, requires less robust ground support, and can be readily excavated using modern tunnel boring machines (TBMs). Highway and rail tunnels in crystalline rock are common in certain regions (e.g., Norway) where they serve for many decades with minimal maintenance. Similarly, openings in volcanic tuff at the Nevada National Security Site that were mined from the 1960s through the 1980s have been stable for decades. The temperature tolerance of crystalline rock is typically higher than clay-bearing sediments (i.e., peak temperature of 200°C for crystalline rock compared with 100°C for clay-bearing sediments), so smaller repository layouts and less tunneling would be needed compared with clay/shale media.

Repository openings in rock salt would not be required to stand open more than a few years according to reference disposal concepts (Hardin et al. 2012) and concepts developed for direct disposal of DPC based waste packages (Hardin et al. 2013). Also, like crystalline rock the heat dissipation and temperature tolerance properties of salt would allow a much smaller repository layout for disposal of U.S. commercial SNF waste, with approximately 1/3 of the tunnel length that would be needed for clay/shale media.

Organization of This Report – The following sections begin with general technical description of excavation methods (drill-and-blast, road headers, and TBMs) and ground support systems (metallic liners, shotcrete, steel sets, rock bolts, cast-in-place concrete, and segemented pre-fabricated concrete) that have been used in clay/shale media. It then describes processes specific to clay/shale geologic settings that are important to long-term tunnel stability and maintenance (response to excavation, shrinkage/swelling, squeezing, ground support interactions, creep, fault deformations, effects from groundwater). This is followed by selected case studies of tunnels in Switzerland, France, Germany, and the U.S. Finally, the discussion and summary section includes a construction/operation scenario description for a repository in an idealized clay/shale medium, for large, heavy DPC-based waste packages.

2. Discussion of Excavation and Liner Performance in Clay/Shale Media

2.1 Excavation Methods

Drill-and-Blast – Blasting methods were used for early industrial-age tunnels, some of which are still in service, but they are not a practical solution for extensive excavation in soft rock and will not be discussed further here. The reasons include speed, cost, rock damage, and worker safety. Drill-and-blast methods remain common in hard-rock (too hard for practical use of road headers as discussed below) especially where the volume to be excavated is too small, or opening shape too complex, to justify mechanized mining.

Roadheaders – So-called roadheaders consist of cutting wheels or bits, faced with hard teeth (e.g., tungsten carbide) spun at the end of a boom. The boom is mounted to a heavy truck and is swept vertically and horizontally across the opening cross-section. Boom movement is actuated by hydraulic cylinders that also maintain the necessary contact force for the cutting wheel on the rock face. Rubble accumulates below the cutting face until there is enough to remove with front-loading equipment (which requires moving the roadheader out of the way). Roadheaders would likely be used in any repository in argillaceous rock for alcoves and platforms, TBM launch chambers, small rooms, etc., even if the preponderance of tunneling is done with TBMs. Roadheaders can also be used in harder rock such as dolomite, welded tuff, quartzose sandstone, etc. but the rate of cutter and machine wear may be significantly increased so that such use is limited.

Tunnel Boring Machines – TBMs are favored for excavation on a scale of hundreds of kilometers because of speed, cost, and minimal rock damage. Modern TBMs may employ advances that include cutters designed for specific rock types, pressurized face machines, improved shields and integrated ground support for soft rock, improvements in muck handling, better control and steering, etc. TBMs have also grown larger with 10-meter diameter now common, and highway tunnels up to 15 meters across excavated in a single pass.

Some different types of TBMs are shown in Figures 1 to 3, and may be characterized as single-shield or double-shield, with or without grippers, and open or pressurized-face designs. The open designs have no excavation chamber or pressure bulkhead, and remove waste rock directly by scooping it onto conveyors, without slurrying. They are used for “hard rock” which may include competent sedimentary rock as discussed below for the Chicago TARP tunnels. Open TBMs may be single- or double-shield designs, with or without hydraulic grippers. Shields stabilize the opening and can serve as anchors propelling the TBM forward. Alternatively, separate grippers may propel the machine, particularly in hard rock, for which a shield may not be needed. The single-shield and/or gripper configurations advance either by pushing against a liner that is erected immediately behind the machine, pushing against grippers set against the wall rock, or where appropriate by pulling ahead (grippers retracted) anchored by the resistance of the cutter head to movement. Double-shield TBMs typically advance by two-anchor (inchworm) locomotion, but may also have grippers for simpler operation in hard rock.

For soft rock pressurized face machines are common, and are either of the slurry shield (SS) or earth-pressure balance (EPB) type. Pressurized-face machines have a pressure bulkhead behind the cutter head, defining an excavation chamber ahead in which the rock is fully exposed. A substantial shield presses against the full circumference of the wall behind this bulkhead, and may also seal against the outer surface of the installed liner behind the shield, to limit groundwater inflow.

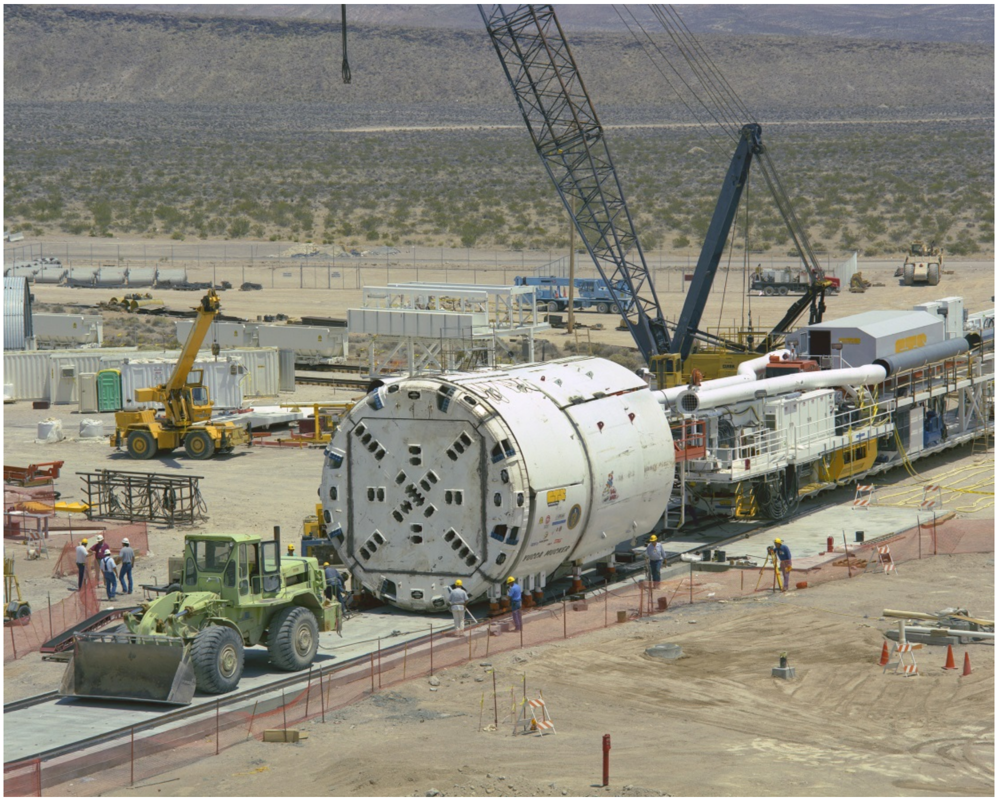

Figure 1. Open-type double-shield TBM with trailing gear, during assembly, Yucca Mountain, Nevada (diameter 7.6 m).

Figure 2. Open-type single-shield TBM with trailing gear, during assembly, Russian road tunnel project (13.2 m diameter) (www.herrenknecht.com).

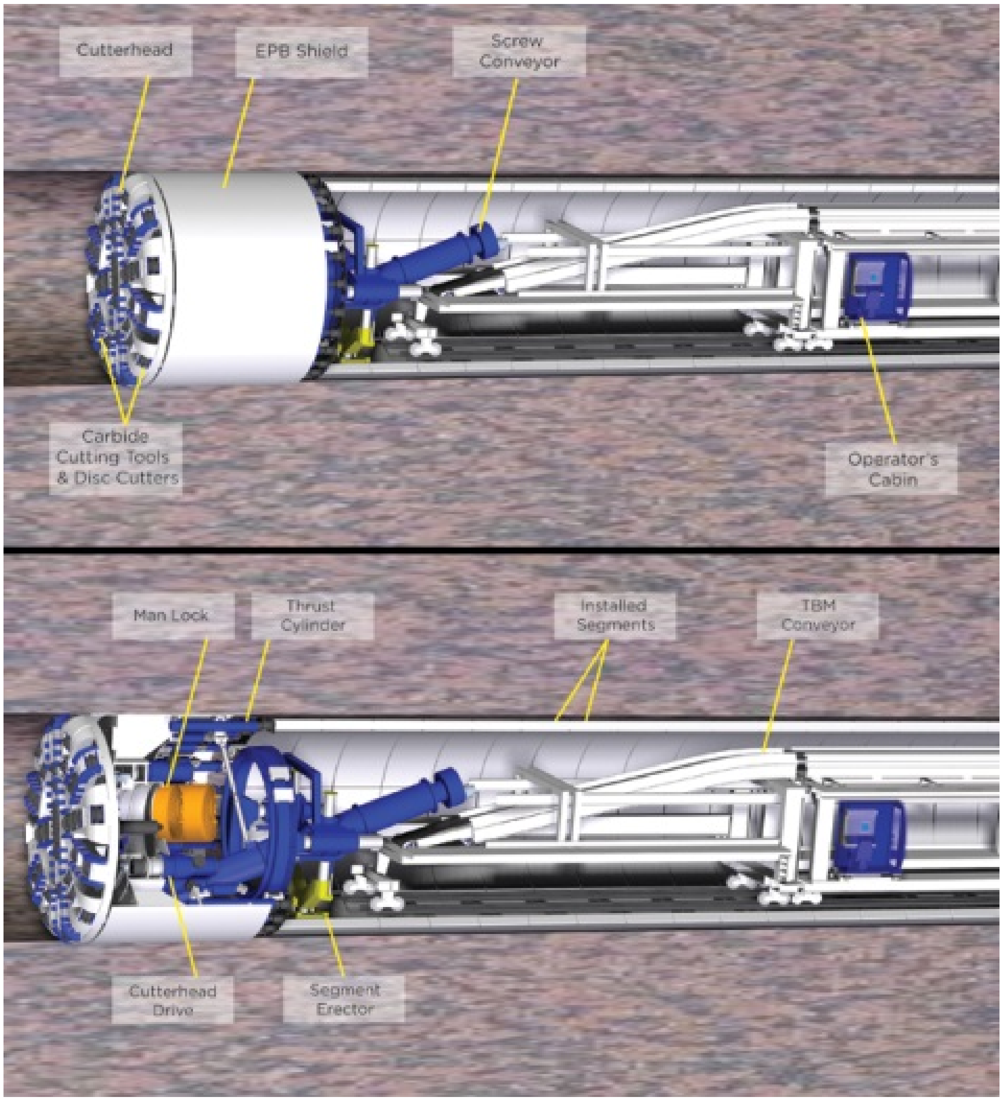

Figure 3. Schematics of an EPB tunnel boring machine showing shield, segmented liner, muck removal system, and other features (www.therobbinscompany.com).

In a slurry shield design, a clay slurry is injected at pressure into the excavation chamber and removed with the cuttings via a pipeline. Slurry plants at the surface separate cuttings material and reinject the slurry.

Earth-pressure balanced machines create a clay-bearing muck from the excavated material, and remove it by a screw conveyor which also controls the pressure. Additives such as clay slurry may also be injected into the excavation chamber. EPB machines minimize deformation but are limited to overburden or water inflow pressures up to approximately 1 MPa. Such shallow applications are beyond the scope of this report, but more information can be found at www.therobbinscompany.com/en/case-study/. EPB and SS machines may have single or double shields, and other dual-mode features that support use of a single machine for a project in variable ground conditions.

Limits on excavation chamber pressures (Figure 3) that can be maintained with pressurized-face machines could preclude their use at repository depths (300 to 900 meters). Major concern with such machines at higher pressures are: 1) the excessive air pressure to which workers are exposed when they enter the excavation chamber for maintenance, and 2) pushing the TBM forward against the chamber pressure. The less efficient but accepted practice of pre-excavation grouting in boreholes drilled ahead of the tunnel face would likely be used instead, possibly with an open-face TBM design. This in turn requires that the rock have a minimum compressive strength on the order of 10 MPa (or greater, commensurate with the combined overburden pressure and hydraulic head) so that the excavation chamber is self-supporting during drilling, grouting operations, and maintenance activities. Pre-excavation grouting may be planned well in advance based on water inflow, permeability testing, and mechanical stability in exploration boreholes. Grouting may not be needed along the full length of tunnels excavated, depending on rock conditions.

TBM trailing or "backup" gear consists of equipment linked to the boring machine, arranged on deck structures that typically roll on temporary rails. This equipment may serve the functions of muck handling or pumping, liner segment handling and installation, liner grouting, installation of the invert and rail, installation of electrical power and lighting, and dust control. Trailing gear may extend for 100 m or further behind the face. TBMs can advance up or down at grades of 5% or greater, but access to trailing gear by conventional rail equipment limits grades to approximately 3% or less. Steeper grades can be accommodated using other means to pull rail-mounted equipment in and out, such as rubber-tire, diesel-powered tugs.

Readily disaggregated soft rock from pressurized-face TBMs may be slurried, pumped to the surface (in stages as necessary), and deposited in tailings ponds. Harder waste rock from open machines is typically removed on belt conveyors. These are also staged to provide conveyance all the way to the surface especially if ramp access is available. Conveyor belts are configurable for curves and ramps, and are also available for vertical transport in shafts. Shaft conveyors are typically used in shallower applications (less than 300 m) but could be staged to handle greater depths.

Repository drift layouts can be designed for TBM excavation by using appropriate turn radii at intersections ("turnouts"). Examples of proposed TBM layouts for different repository concepts are given by Hardin et al. (2012). The goals of layout design are to prevent or limit any need for disassembly and relocation of the TBM and trailing gear, either for maintenance or by design. Instead, layouts would have loops that allow the TBM breakthrough to a previously excavated opening, and movement in the forward direction to the location of the next heading to be mined.

This is a brief summary of excavation technologies with emphasis on major types of TBMs. Much more information can be obtained from TBM contractor-vendors, tunneling journals (e.g., *Tunnelling and Underground Space Technology*) and manuals developed by agencies and industry associations (see Section 2.4).

2.2 Liner Types and Degradation Modes

Liner longevity is a key issue in long-term performance or repository drifts in clay/shale media. Liner materials and dimensions must be selected to bear the loads applied by the host medium, for at least 50 years in the repository environment (oxidizing, and ranging from wet to dry at elevated temperatures up to 100°C). In clay/shale media that are overconsolidated and/or have the potential for swelling, the liner provides an isolation barrier to moisture or groundwater movement. In all clay/shale media, the liner provides confinement during and after initial deformation.

The earliest liners for tunnels in clay/shale media were masonry, typically for rail and canal tunnels. Mortared brick or stone masonry had the advantages that minimal mechanization was required for construction, and portions could be easily replaced. However, construction was slow and thicker masonry was needed as the required opening spans increased. Other methods and materials were eventually adopted for tunnel construction, but it is interesting to note that many older masonry-supported tunnels are still open in the U.S. and elsewhere, in shale and other soft sedimentary media. Many older tunnels are located in Pennsylvania and other mountainous areas of the Eastern U.S. (e.g., Staple Bend and Big Savage; NPS 2007).

The design life of modern liner systems may range from a few years for underground exploration and research, to more than 50 years for highway, railway and water conveyance projects. The range of design solutions varies accordingly, and includes shotcrete, steel or cast iron liners, steel supports, segmented pre-fabricated concrete, and cast-in-place concrete:

- **Shotcrete –** Generally consists of Portland cement mixed with sand, with water typically introduced at the spray nozzle. Plasticizers are used to decrease the water:cement ratio and thus increase strength. Glass or metal fibers may also be introduced to provide reinforcement. May be used over a layer of wire mesh or fabric bolted to the rock surface. Layers of 5 to 10 cm thickness are typically applied, and may be applied repeatedly to achieve 30 to 50 cm total thickness.

- **Steel or cast iron –** Used for immediate ground support in potentially unstable (e.g., squeezing) ground. For small openings pipe or tubing may be used, while for larger openings the liner may be assembled in segments bolted together. Used for initial construction at the underground laboratory in the Boom Clay at Mol (ONDRAF/NIRAS 2001); later construction used a liner of pre-fabricated concrete segments.

- **Steel ribs or sets –** A method readily used for remedial support of weaker sections. Steel ribs are blocked against the wall and roof using shotcrete, steel, wood, etc. Lagging consisting of wooden or steel panels may be laid between steel arches, which are typically flanged. Ribs may be bolted to the walls for additional support, particularly at the springline. Backfilling with additional lagging or shotcrete can be used to fill gaps between support elements and host rock. Steel arches may be: 1) fully circumferential, or anchored in concrete or steel invert elements; 2) may be used in conjunction with wire fabric, and a layer of shotcrete applied between sets (e.g., for the URL at Bure, discussed below); 3) may be used with lattice girders to support larger spans at intersections; and 4) may also be used with cast-in-place concrete to provide initial support that is later encapsulated (e.g., for the Azotea Tunnel discussed below).

- **Segmented pre-fabricated concrete** - The most common support method currently used for TBM-excavated tunnels. Liner segments may be 1 to 2 m high (circumferential), 2 to 3 m long (axial), and 0.3 to 0.75 m in thickness (radial). They generally contain reinforcing steel in some form, and may be fabricated from very high strength concrete (e.g., 70 MPa or greater). Segments may have hollow form or uniform thickness. Their shape and size are amenable to automated handling and installation. A ring of segments may be completed using a wedge segment that can be expanded to generate arch loading. Segmented liners are typically backfilled with grout by pumping through ports in the segments. The grout may be compliant (e.g., with aggregate of plastic beads or foam particles) to accommodate large, localized deformations of the rock wall while maintaining compressive loading in the liner.

Two types of segmented liners are used: 1) gasketed, bolted segments with backfilling comprise a single-pass, final liner (Figure 4); and 2) ungasketed, unbolted arch segments with light reinforcement, with backfilling, followed by another smooth liner (typically 20 to 30 cm of unreinforced concrete). The first type is more common, and may be used with doweled circumferential joints (Hung et al. 2009, Section 8.3.6).

- **Cast-in-place concrete –** Unreinforced concrete was used extensively in 20th century tunneling, and is still commonly used for shafts. Design thickness of the concrete may range from 0.2 to 0.75 m depending on rock characteristics and *in situ* stresses. Cast-in-place concrete is relatively expensive because of the costs of form setup, mixing or transporting uncured concrete underground, and the likely delay of excavation during initial cure.
- **Shotcrete –** A wide variety of shotcrete materials and reinforcing fibers, with ranges of strength, adhesion, mechanical resistance, etc., and applied at different thicknesses for various applications, is reported in the literature (Franzen et al. 2001). It is possible that construction of a repository with hundreds of km of drifts would make extensive use of shotcrete because of its low cost compared to steel and cast-in-place forming. Shotcrete could aso be significantly cheaper than pre-fabricated liner segments, and suitable if greater strength is not needed.

Liner design for a repository must consider short-term and long-term interactions with the host medium, over the operational lifetime of the facility. This means the liner will likely be subjected to additional, long-term loading if significant creep occurs in the surrounding rock (see discussion of host rock responses below). Assuming that liner elements are correctly specified to meet loading conditions, and that the liner is sufficiently impervious to prevent shrinkage/swelling of the host medium, then maintenance issues could be limited to degradation of liner materials. Concrete can have a lifetime of 50 years especially if unreinforced (see discussion of the San-Juan Chama Project tunnels) while performance of shotcrete in thinner layers depends on stability of the underlying rock, and bonding strength (see discussion of the Mont Terri URL tunnels). Temporary support measures are less substantial and may loosen or fail in fewer than 50 years. Ultimately, ground support failure can result if the underlying host rock fails, for example from long-term deformation at interfaces or due to rock creep.

Figure 4. Pre-cast reinforced concrete liner segment, removal from forms, for a single-pass liner (Hung et al. 2009, Figure 10-7).

2.3 Host Rock Response to Excavation

The initial response to excavation is redistribution of stress from the excavated volume into the surrounding medium. This process concentrates stress near the walls, roof and floor of the opening. Whereas the initial *in situ* stress condition in the host rock may be nearly lithostatic, the immediate stress condition adjacent to mined openings is strongly deviatoric (i.e., large differences in stresses depending on direction). Deviatoric stresses can produce large deformations, dilatancy and fracturing especially in media with low strength compared to the overburden stress. For example, a soft shale unit with unconfined compressive strength of 5 MPa will undergo these effects when excavated at a depth of 300 m (7 MPa overburden stress). Strength increases with confinement, which occurs with increasing distance from the opening, and also occurs after liner installation.

Much of the immediate deformation associated with excavation occurs in the region ahead of, or close to the active mining face. Thus, stress redistribution is inherently three-dimensionsal and produces three dimensional deformation features, where concentrated stress magnitudes exceed strength criteria. Dilatancy or bulking of the medium occurs when shear stress exceeds strength limits, and typically leads to weakening and plasticity. Pore fluid pressure release near an excavation may contribute to failure by pore-scale transfer of load from fluid pressure to the solid framework. Fractures form along surfaces of low normal stress and high shear, and parallel arrays of *en echelon* fractures form near the face as the tunnel is excavated. These excavation-induced fractures may be evident as “chevron-like” shear fractures where they intercept the roof or walls of the tunnel (Delay et al. 2010). In laminar, anisotropic media these processes interact with pre-existing planes of weakness. The resulting fractures that form near the working face determine the extent of the excavation damage zone (EDZ) around every underground opening.

Another important type of fracturing is “slabbing” that may occur in the walls or roof, caused by slippage on shear fractures that form close to the opening where concentrated, deviatoric stress is greatest. In relatively young sediments within a few hundred meters of the surface, the maximum stress is likely to be vertical so that failures form on the walls. In rock with “shaly” or laminated fabric, slab failures may be evident in the roof due to weakness or separation along bedding planes. The liner functions to confine the rock near the opening, increasing friction along potential failure surfaces. Degradation of liner properties or separation from the rock wall can contribute to rock failure where the maximum compressive stress exceeds the unconfined compression strength.

Clay-rich media can be sensitive to moistening or drying which causes swelling and shrinkage, respectively. This potentially destabilizing behavior has been understood for many years in soft-rock mining, and referred to as slaking. It can be serious when groundwater that flows into mine openings interacts with exposed sensitive rock surfaces. Equivalent behavior may also occur with exposure to changes in humidity. Formation of shrinkage cracks during the winter, and closure of the cracks when humidity increases in summer, has been observed and monitored at the Tournemiere URL (Pellet 2014, verbal communication). In repository drifts heated by SNF waste, severe drying conditions will occur which could produce penetrating shrinkage cracks that destabilize exposed surfaces of the host rock. Also, repeated shrinkage and swelling due to fluctuations in ventilation conditions could cause destabilizing fatigue. Therefore, an important function of the liner in clay/shale media is to isolate the host rock from open air spaces, especially heated and/or ventilated spaces.

Squeezing behavior is progressive deformation that starts immediately after excavation and can produce large opening closure deformation within days to months. Squeezing deformation commonly occurs in the invert, and is also common in crossing faults (Hung et al. 2009, Sections 8.2.5 and 8.3.4). Squeezing is time-dependent, with the rate limited by frictional processes acting within the medium, and therefore may act as a form of creep. In mining applications squeezing may be ignored if it does not interfere with the completion of extractive activities. In repository applications squeezing is potentially beneficial as it could eventually close and reseal repository openings. However, during excavation, construction, and preclosure repository operations it must be controlled. Slower long-term creep could also be important within the 50-year timeframe of repository operations. One approach to managing squeezing ground and creep is to use yielding supports, designed to crush or collapse while providing constant support (Hung et al. 2009, Section 8.3.3; also see Section 3). Squeezing and creep may also be controlled by installing a liner robust enough to assume enough of the deviatoric loading in the host rock, to slow or stop the deformation. This process is illustrated by calculations for the Pierre Shale (Figures 5 and 6) which show how deviatoric stress around a circular opening could relax over 10 to 20 years, as creep occurs and load is transferred to a robust liner of high-strength concrete (Nopola 2013).

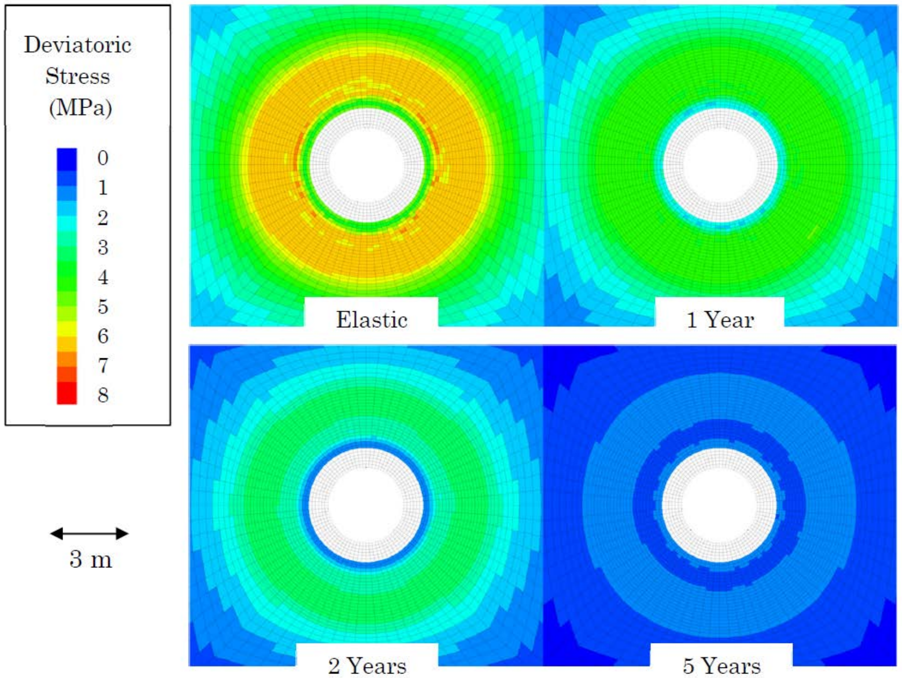

Figure 5. Viscoplastic solution to stress distribution in the Pierre Shale, around a 3-m (finished) diameter circular tunnel at 700 m depth, showing load transfer to a 0.75-m thick concrete liner (with permission, Figure 4-12 from Nopola 2013).

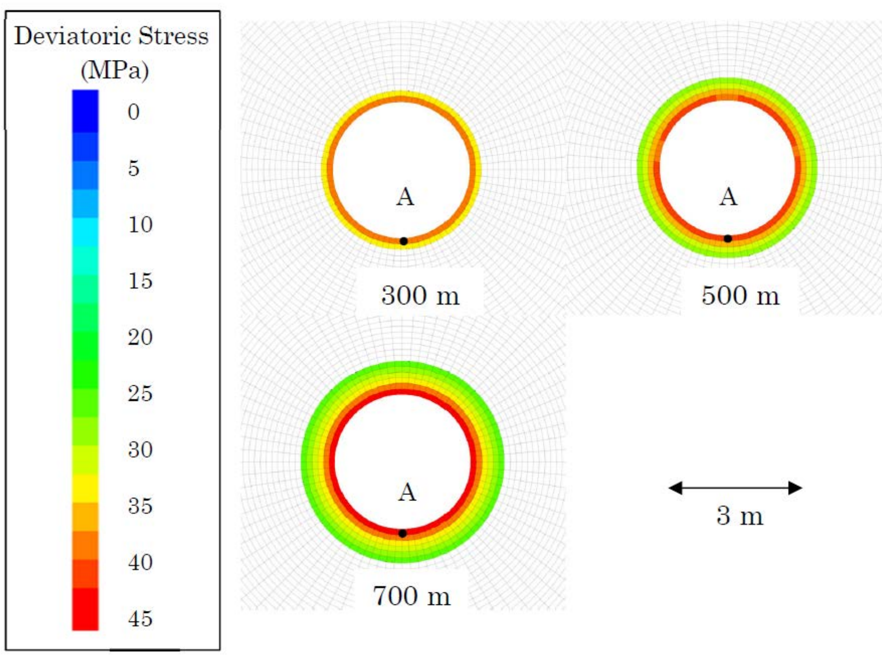

Figure 6. Concrete liner stress after 20 years of load transfer, for a 3-m (finished) diameter tunnel in the Pierre Shale, at the depths indicated (liner thicknesses of 0.25, 0.50 and 0.75 m; with permission, Figure 4-8 from Nopola 2013).

Groundwater can impact opening stability by causing water inflow, and by hydraulic loading of the liner. In potential clay/shale host media these processes are not expected to be important because: 1) although the host rock may be nominally saturated, the bulk permeability will be low; and 2) water-bearing faults can be characterized, sealed by grouting or other means, and isolated from repository openings. Groundwater could be important in excavations that must penetrate non-host stratigraphic units such as aquifers that overly the host rock. However, the measures needed to mitigate such an occurrence would be site specific, and are beyond the scope of this report.

Most sedimentary basins contain faults, which may be active or remnants of previous tectonic activity. Relatively recent processes such as loading from continental glaciation may also have produced faults. The mechanical effects from faults on excavations that intercept them could dominate the long-term stability of nearby repository openings. This is because fault zones are fractured, sheared, chemically altered, and may be pathways for groundwater flow. Faults that offset the host rock strata may cause mixed geologic conditions at the excavation face, which complicates mining and construction. In general, repository tunnels could intercept faults with unfavorable geometry (e.g., with parallel strike, or hanging blocks in the walls or roof). Faults are routinely intercepted by tunnels such as those described in the case studies below, so moderate faulting is not an acute concern for repository construction feasibility. Pre-excavation borehole grouting is commonly used to remediate faulted ground for tunneling. However, fault characterization is needed and the repository layout and concept of operations should accommodate fault characteristics. To support this goal a minor fault ("Main Fault") at the Mont Terri URL is being investigated by the PS-Experiment and the planned water injection characterization experiment (B. Laurich/Y. Guglielmi 2014, verbal communication).

2.4 Rail and Highway Tunnel Design Guidelines

Compendia describing technologies for underground tunneling and construction are available from government and other sources. A recent contribution is the Federal Highway Administration's *Design and Construction Manual for Road Tunnels* (Hung et al. 2009). This manual summarizes conceptual cost analysis, groundwater control, maximum grades, and geotechnical investigations. It provides a guide to TBM selection, considering: whether formation conditions are firm, raveling, flowing, squeezing, or swelling ground; weak or hard rock; permeability and groundwater head; presence of gas, boulders, etc. The FHA guide describes ground support options for adverse rock conditions, seismic design, evaluation of constructability, staging, scheduling, and geotechnical instrumentation.

3. Selected Case Studies

The bulk of this report is devoted to case studies of modern, large-scale tunneling projects. Some of these are in clay/shale media, including some with overburden depth in the range 300 to 900 m that would be expected for a repository. Also included are large and/or famous projects that provide context for discussion of engineering solutions and costs. A summary of tunnel project metrics is provided in Table 1.

Tunneling activities in the Opalinus Clay, Callovo-Oxfordian argillite, Boom Clay, and the Pierre Shale are of particular significance because these are currently being studied as potential repository host media, or have been investigated in the past. Tunneling examples in these media therefore have particular relevance to generic studies of the repository construction for disposal of large, hot waste packages. The following descriptions of these rock types are summarized from Hansen et al. (2010).

The Opalinus clay is an indurated claystone that is common in the Jura region of northwestern Switzerland, where its thickness is 50 m or more. It is a bedded, stiff, Mesozoic claystone of marine origin. The Opalinus formation is composed of 50% to 65% clay (Table 2) mostly kaolinite and illite. The response to deformation is generally transversely isotropic and brittle, although creep has been observed *in situ*. Strength of the Opalinus clay allows small, unlined tunnels and larger, lined tunnels to be constructed at depths of several hundred meters. When subjected to heating, the Opalinus in its natural state exhibits a strong pore pressure response that affects its hydraulic and mechanical behavior.

The Callovo-Oxfordian (COX) argillite is slighly more indurated (i.e., lithified) than the Opalinus, but is of similar geologic age and provenance. It has a thickness of approximately 130 m at the URL site (185 km straight-line distance from Mont Terri). Reported average compositions vary with respect to percentages of carbonate, clay, and quartz and feldspars (Table 2). Creep behavior in the COX argillite has been observed (Section 3.3). A North American shale formation, the Pierre Shale, was characterized previously as a potential host repository medium (Neuzil 2013). It is of younger geologic age than the Opalinus or COX formations, and is less indurated. It occurs throughout the northern Great Plains, often at shallow depth. Substantial ground support could be needed at depths of hundreds of meters where the overburden stress exceeds rock strength (Section 3.4.5).

By contrast, the Boom Clay is of younger geologic age, and is plastic (i.e., not lithified). The porosity of the Boom clay, and hence its water content, is approximately 20% to 30% by volume (Table 2). Excavation is relatively easy in this soft material, but tunnels at repository depth would likely require immediate, heavy lining to stay open during construction and repository operations. Plastic clay has certain advantages as a repository host medium, but feasibility of underground construction is likely not among them. Although plastic clay could be an option for the U.S. repository R&D program, it is not covered in this review.

Table 1. Comparison metrics for tunneling projects discussed in this report.

Project [A]	Country	Lithology	Depth (m)	Excavation Method	Ground Support	Diameter (m)	Length (km)	Cost	Escalated Cost/meter [B]
St. Martin La Porte access adit	France	Various (Carboniferous)	<2,500	Drill/blast and road header	Rock bolts+steel ribs+ shotcrete+deformable elements	6 to 9	2.3	NA	NA
Channel Tunnel	France-Britain	Chalk	≤100	TBM	Pre-fab. segmented concrete liner	5.3 to 8.3	50 (×3)	$7B (1988)	$100k
SJCP Azotea Tunnel – New Mexico	USA	Mancos & Lewis Shales/ other sediments	≤488	TBM	Rock bolts+steel ribs+ cast-in-place concrete liner	4.3	20.6	$17.1M (1989)	$1.7k
SJCP Blanco Tunnel – New Mexico	USA	Lewis Shale	≤600	TBM	Rock bolts+steel ribs+ cast-in-place concrete liner	3.1	13.9	$9.8M (1989)	$1.5k
SJCP Oso Tunnel – New Mexico	USA	Shale/glacial debris	≤232	TBM	Rock bolts+ cast-in-place concrete liner	3.1	8.1	$5.8M (1989)	$1.5k
Phase 1 TARP – Chicago	USA	Niagara Dolomite	<100	TBM	Rock bolts+ cast-in-place concrete liner	10.8	176	$4B (2006)	$29k
Euclid Creek Storage Tunnel – Cleveland	USA	Chagrin Shale	~60	TBM	Pre-fab. segmented reinforced concrete liner	8.2	5.5	$200M (2014)	$36k
Mill Creek Phase 2 – Cleveland	USA	Chagrin Shale	49 to 79	TBM	Steel ribs+lagging+ cast-in-place concrete liner	6	4	$57M (1999)	$22k
Mill Creek Phase 3 – Cleveland	USA	Chagrin Shale	63 to 93	TBM	Steel ribs+lagging+ cast-in-place concrete liner	6	4	$73M (2002)	$26k
Oahe Dam tunnels – South Dakota	USA	Pierre Shale	~100	TBM	Pre-fab. segmented concrete liner	9	14 (total)	NA	NA
Flathead Tunnel – Montana	USA	Quartzite/argillite	≤200	Drill/blast	Steel ribs+lagging	~7	11.3	$37.9M (1969)	$13k
Niagara Tunnel #3 – Ontario	Canada	Queenston Shale	≤140	TBM	Rock bolts+wire fabric+ shotcrete+ cast-in-place concrete liner	14.4	10.4	$3.4B (1999)	$500k
Park River Tunnel – Connecticut	USA	Shale/sandstone	60	TBM	Pre-fab. segmented concrete liner	~7	2.8	$23.3M (1978)	$24k
Plateau Creek Tunnels – Colorado	USA	Sandstone/shale/ siltstone	?	TBM	Rock bolts+wire fabric+shotcrete	3.3	4.1	$14.1M (2001)	$5k

Notes: [A] For sources see references in text of Section 3. [B] Escalation factor 3% per year to 2014.

Table 2. Properties of well-characterized clay/shale media (after Hansen et al. 2010)

Shale Formation	Reference Location	Approximate Geologic Age (Ma)	Typical Thickness (m)	Top Burial Depth Present/Past (m)	Clay Content (wt. %)	Classification[1]	Mineralogy[2]	Carbonate Content (wt. %)	Hydraulic Conductivity (m/sec)	Compressive Strength[3] (MPa)	Organic Content (wt. %)	*In situ* Water Content (vol. %)
Europe:												
Opalinus Clay	Mont Terri, CH	180	160	250/1350	50 to 65	Claystone	Kaolinite, illite, illite/smectite	10 to 50	Est. 5×10^{-13} to 6×10^{-14}	12	0.5	4 to 6
Callovo-Oxfordian Argillite	Bure, France	155	130	400/NA	45	Mudstone	Illite/smectite	20 to 30	Est. 3×10^{-14}	25	< 3	5 to 8
Boom Clay	Mol, Belgium	30	100	220/NA	55	Bedded mud	Smectite/illite	1 to 5	Est. 6×10^{-12}	2	1 to 5	22 to 27
North America:												
Pierre Shale	Pierre, SD	70	400	150/NA	50	Mudstone	Illite/smectite	0 to 50	10^{-13} to $10^{-14.6}$	7	0.5 to 13	~16 (variable)

Sources: ANDRA 2005; Hansen and Vogt 1987; NAGRA 2002; NEA 2003; Neuzil 2000; Volckaert et al. 2005.

Notes: [1] Use clay-mud-claystone-mudstone-argillite classification from OECD/NEA 1996, p. 4. [2] Predominant assemblage or combination: smectite, illite, kaolinite, chlorite, carbonate, etc. [3] Unconfined, typical laboratory values for fresh samples. NA = not applicable (past burial depth not significant).

3.1 Switzerland

Highway and rail tunnels are especially abundant in Switzerland because of the populated mountainous terrain, and the practice of high technology. The construction of long, deep rail tunnels began in the mid-1800's (as in the U.S.) and some of these tunnels have been in service for more than 100 years. A summary of early Swiss tunnels in the Opalinus Clay is provided by Einstein (2000) and the topic will not be addressed here because drill-and-blast excavation and masonry liner construction would be uneconomic choices for a repository.

The focus of this survey of Swiss tunnels is modern, TBM-excavated rail and highway tunnels in the Jura region of northwestern Switzerland, and the Mont Terri URL in the Opalinus Clay. A few representative tunnels are discussed to give an overview of the state-of-the-art and how it has developed over the past few decades.

Tunnels in the mountainous Folded Jura tend to be deep, with clay/shale exposures at depths of 300 m or more common. In the flat-lying Tabular Jura to the northwest there are far fewer existing tunnels, but this is an area being considered for siting of a geologic repository at a depth of roughly 800 m in the Opalinus Clay. Construction of stable openings in clay/shale media at this depth is an important technical issue for the Swiss repository program. A Swiss national symposium on tunneling in claystone was held on February 14, 2014 at the Zurich Technical University (ETH). A summary of presentations given at that event is included as Appendix A.

3.1.1 Highway Tunnels in Northwestern Switzerland

Early-modern tunneling methods in Switzerland were distinguished by use of movable excavation shields to stabilize the rock before liner installation (Steiner 2014, verbal communication). The earliest methods used drill-and-blast excavation with a shield, and a segmented liner (e.g., pre-fabricated concrete or cast-iron). For example, the Baregg Tunnel (A1 motorway, tubes 1 and 2) completed in 1970, was constructed by drill-and-blast with a horseshoe-shaped shield. Non-circular shields were found to be unworkable because they have a tendency to roll and cannot be readily corrected or steered.

The Heitersberg Tunnel (single-track rail) was constructed in 1970 using a Robbins open-type TBM (10.8 m diameter). Ground support consisted of an outer liner of shotcrete sprayed by a robot attached to the TBM, with wire fabric and rock bolts, and steel supports where needed (24% of total length). The Gubrist Tunnel (motorway tubes 1 and 2), completed in 1985, was excavated using a similar arrangement and the same shield, and the liner was mated with the shield to improve the stability of the interval between them. The Rosenberg Tunnel, another early-modern highway tunnel, was excavated using a shield with four road headers.

Soft-rock tunneling in Switzerland advanced after the 1970's, accelerated by extensive tunnel construction using TBMs, for the A16 and other motorway routes (Steiner 2014, verbal communication). For examples, the Mont Russelin highway tunnel completed in 1998 is 3.5 km long, with more than 300 m of maximum overburden. The Bözberg twin tubes (A3 motorway) were completed in 1994, and are 4.3 km long with more than 200 m maximum overburden. The Adler rail tunnel was completed in 2000, is 5.2 km long, and was excavated to a diameter of 12.5 m (then a TBM record). The Bure highway tunnel was completed in 2011, is 3.1 km long, and has a diameter of 12.6 m. The Mont Terri highway tunnel was completed in 1998, is 4 km long, and replaced the tunnel now used to access the Mont Terri URL.

Tunneling in the Opalinus Clay has achieved good results with single-shield TBMs, avoiding the problems associated with conventional methods such as drill-and-blast. Such tunnels have circular cross-section, and a continuous segmented liner installed immediately to prevent swelling. Stability problems close to the face may be encountered, especially in fault zones, and where there is strong water inflow. Rock instability in the crown can be mitigated by installing deformable filler material (e.g., pea gravel) behind or above the segmented liner, while support can be increased by injecting grout into the filler (30% to 50% porosity). With application of these methods since the 1970's, advance rates have improved, with larger excavated opening diameters.

Einstein (2000) described a series of experimental and analytical studies sponsored in the 1990's by the Swiss Federal Office for Road Construction, and the Swiss Federal Railroads. These studies comprised laboratory measurements to support derived constitutive models for anisotropic, elastoplastic deformation with poroelastic coupling (permeability controlled). Numerical simulations were presented for circular tunnel diameters of 2.4, 5.5 and 6.2 m; with horizontal/vertical stress ratios K = 0.5, 1 and 2 (σ_H/σ_V); burial depth of 400, 700 and 1,000 m; and liner thickness of 0.2 and 0.3 m.

From these results a general prescription for stable tunnel design in the Opalinus emerges (from Einstein 2000):

- Circular tunnel with circular liner of uniform thickness;
- TBM or other mechanized excavation methods, to minimize rock damage and produce smooth-walled openings;
- Initial liner consisting of pre-fabricated, high-strength concrete segments, produced and installed with high quality control;
- Protection of the tunnel invert by liner installation at a relatively short distance behind the face, so that construction water and groundwater inflow do not contact exposed rock; and
- The final liner (grout-backfilled segments, possibly with cast-in-place concrete inner liner) is watertight.

Analyses show that immediate installation of support causes the greatest tangential normal stress in the liner (e.g., more than 80 MPa at 700 m depth), but that delayed liner installation decreases support loads by more than half. Significant anisotropy should be factored into design, analysis and testing. For example, mechanical anistropy causes the liner stress for K=0.5 to exceed that for K=2, and it increases maximum liner stresses for lithostatic stress conditions (K=1). Higher permeability allows more drainage after excavation, which increases the magnitude of deformations during the delay before liner installation. Conversely, lower permeability can increase support loads signficantly. Analysis showed that unsupported Opalinus Clay may fail as it is being excavated, as excavation stress paths tend to approach modeled strength limits.

Another survey (Nater 2014, verbal communication) identified four excavations from which tunneling experience in clay/shale media can be drawn: the historic Grenchenberg tunnel in Switzerland, the Mont Terri URL, excavations at the Konrad repository in Germany, and the Bure URL. The Grenchenberg single-track rail tunnel was completed in 1916 with a length of 8.6 km. It traverses the Opalinus claystone at approximately 700 m below the ground surface. Rock conditions in the Opalinus were reported as good, and dry (unlike water inflow zones encountered elsewhere), but support requirements were relatively high. The lining consisted of vaulted masonry up to 60 cm thick, and the invert was constructed as an arch with similar thickness. The tunnel was eventually abandoned after some 50 years of service. There are many tunnels through the Opalinus in Switzerland, and this one may be unique because of its length, depth and age. The other tunnels discussed by Nater (see Appendix A) are discussed in the following sections.

Survey of Swiss tunnels would not be complete without mention of the Gotthard Base Tunnel, currently in the final stages of construction. The project consists of twin single-track rail tunnels, excavated diameter 8.8 to 9.5 m, and 57 km long making it the longest in the world (Alptransit 2010). Two parallel shafts and three side tunnels connect the tunnels to the surface at four locations, in addition to the north and south portals. The tunnel is constructed in metamorphic and igneous rock, and the overburden depth varies up to 2,500 m, with nearly 100% of the tunnel under at least 1,000 m. Drill-and-blast methods were used for approximately 44% of the excavation, while open gripper-type TBMs were used for the other 56%. Tunneling began in 2003 and was completed in 2011. The greatest difficulty, and the reason for so much conventional drill-and-blast excavation, was highly stressed squeezing ground encountered along several sections. Also, strong water inflow in certain sections required extensive borehole grouting (Ehrbar 2008). Rail service is scheduled to begin in 2017. A similar twin-tunnel project is under construction in Italy, called the Ceneri Tunnel, which will link the Gotthard line from Zurich to Milan.

3.1.2 Mont Terri

The Mont Terri tunnels are designed for R&D, with a lifetime on the order of 20 years, with only local repairs needed (Bossart 2014, verbal communication). Immediately after excavation a thin layer (5 cm) of shotcrete is applied to prevent hydration or slaking. All shotcrete is formulated for early strength and low pH (balanced portlandite, excess soluble silica, and super-plasticizer). Wire fabric (e.g., coarse “welded wire cloth”) is then installed with short bolts. Long rock bolts may then be installed for larger openings, or where dictated by performance requirements (e.g., to minimize maintenance in certain parts of the URL). Rock bolt length is typically 100% to 150% of opening diameter, and they are installed using a full 270° pattern. Bolts may be metal or fiberglass, and may use point anchors or full grouting. Alternatively, steel sets have been installed for additional support at some locations, with “distortion zones” at the mid-height of the tunnel walls (i.e., tunnel springlines).

After a delay of several months to allow convergence, a final layer of shotcrete is applied (15 cm). As much as 5% convergence has been observed where stress is concentrated by nearby excavations (e.g., the FE test alcove). The floor is shotcreted all the way to the ribs, and the surface of the shotcrete is worked to provide a running surface for water. Drift convergence tends to continue until the floor is shotcreted. At Mont Terri, a few percent of the total drift length has been impacted by shotcrete slabbing on the ribs, which occurs in these locations with a frequency of 10 to 20 years (Figure 7). Renovation consists of removal of loose shotcrete and rock using a hydraulic hammer, replacement of wire fabric, and application of new shotcrete. This behavior is attributed to creep, tensile stress-induced fracture, and de-bonding of the shotcrete (Bossart 2014, verbal communication).

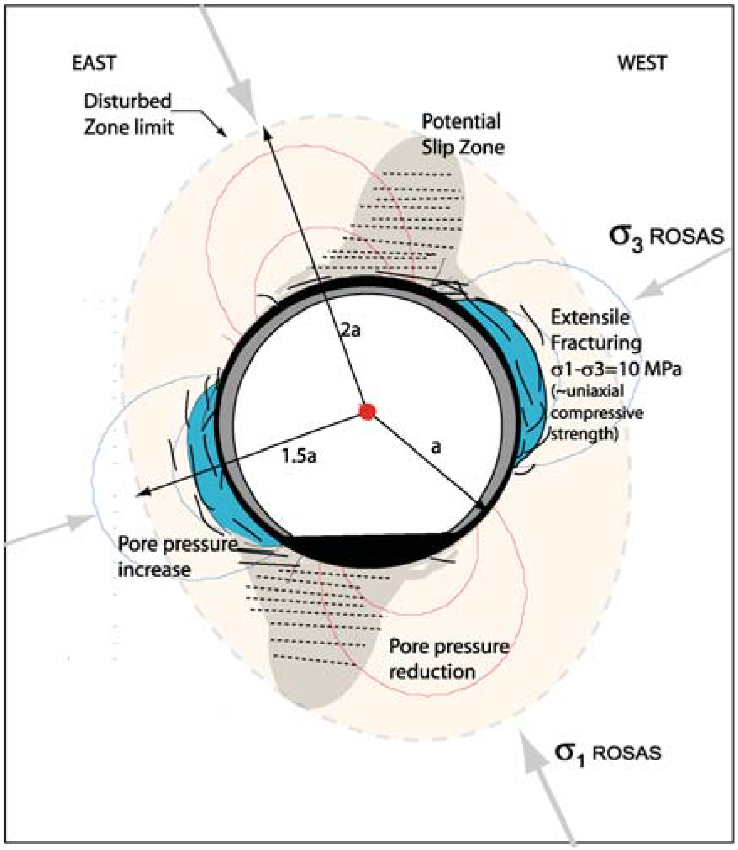

Figure 7. Schematic of slab-type failures that occur sporadically in the URL tunnels at Mont Terri (overburden depth 250 to 400 m) (from Bossart et al. 2014).

Rock damage at Mont Terri is dominated by bedding-induced breakouts. Stress-induced breakouts are uncommon (at the 250 to 400 m overburden depth). Some of the drifts aligned parallel to the strike of bedding have experienced shear zone failure, with large-block (e.g., 1 to 2 m thick) movements detected using borehole extensometers. The EDZ at Mont Terri typically exhibits characteristic *en echelon* fractures ("plumose hackles"of Martin and Lanyon 2002) and the mechanism may be related to pore pressure excursions. Hydraulic conductivity of the EDZ at Mont Terri is on the order of 10^{-8} m/sec, and gradual reduction is observed. For a deeper repository in the Opalinus (e.g., up to 800 m) the intensity and extent of the EDZ could be significantly greater.

3.2 Germany - Konrad

The Konrad repository is under construction at a former iron mine in central Germany, and is expected by 2019 to begin nominally non-heat generating wastes from medical and research sites, power plant operations and decommissioning, and nuclear fuel cycle activities. The construction at Konrad is included in this review because part of the supporting underground facilities is being constructed in the Lower Cretaceous clay/shale layer that provides hydrogeologic isolation to the host rock below. The repository depth is approximately 1,000 m, and openings in the clay are being constructed just above.

Excavations in the clay/shale will provide access to the repository disposal drifts, from existing shafts constructed originally for iron ore mining. Clay/shale layers overlie the host rock, which is the original “iron ore” (ferriginous oolitic limestone). Stability of openings in the “iron ore” formation is known from the history of mine openings dating back to the 1950’s. Emplacement drifts are currently being excavated using road headers, and support using rock bolts with mechanical anchors, and wire fabric. These drifts will be loaded with packaged wastes, backfilled, and plugged.

Repository service openings are being constructed in the clay/shale layer with design life of 50 years with minimal maintenance. Excavation is being performed using road headers, starting with a top heading then cutting the floors deeper in successive passes, so that roof support and overhead services such as lighting can be installed first. Initial ground support consists of welded wire fabric bolted to the rock, then 8 to 10 cm of shotcrete applied using masks to produce linear gaps in the shotcrete every few meters (Figure 8). This is followed by long rock bolts of 12 to 18 meter length, fully grouted, and 50 to 70 cm apart (spaced closer at pillars). The initial shotcrete forms yielding supports, and the openings (8 m span) are expected to close 30 cm six months. After that time the closure rate will slow down, and the ground support will be completed with 30 to 50 cm additional shotcrete.

Figure 8. Yielding initial support (wire fabric, shotcrete and long rock bolts) just after installation in the Lower Cretaceous clay at the Konrad repository site (depth ~1,000 m; photo by author).

3.3 France

Eastern France shares much of the same geologic history as northwestern Switzerland, including the occurrence of sedimentary basins of Mesozoic age containing thick argillaceous intervals. The following examples include the URL at Bure, where conditions for the Cigéo repository project are being evaluated, and two modern large-scale rail tunnel projects.

3.3.1 Bure Underground Research Laboratory

Design of tunnels for the Cigéo repository will be similar to the URL at Bure. The French repository is required to facilitate retrieval for at least 100 years, which means operational areas must remain stable for up to 150 years (Armand 2014, verbal communication). The disposal concept for HLW will emplace the glass-waste pour canisters directly into long, horizontal borings excavated using a remotely operated mini-boring machine, and lined with continuous steel casing.

Vault-type rooms for low- and intermediate-level waste will be 9 to 11 m wide and up to 400 m long (ANDRA 2005). Disposal rooms and access tunnels will likely be excavated using road headers because of the many different configurations planned, and limits on the scale of tunneling required. Several support options have been identified: 1) yielding support, with 3-m rock bolts and 8 cm of fiber-reinforced shotcrete; 2) resistant support, combining the yielding elements with an additional 27 cm of cast-in-place, unreinforced concrete; and 3) maximum support, combining the yielding elements with 45 cm of fiber-reinforced concrete applied in four layers. Access drifts designed for long-term service could also be supported in a manner similar to tunnels in the URL, which are lined with steel ribs and lagging, with shotcrete emplaced between the ribs (Section 2.2).

Reported *in situ* stress conditions at the Bure site are somewhat different from the Swiss Jura, with unequal horizontal stresses ($\sigma_H/\sigma_h \cong 1.3$) that are greater in magnitude ($\sigma_V \cong \sigma_h$). *In situ* stress conditions at depth in the Tabular Jura are expected to be nearly lithostatic (i.e., $\sigma_H = \sigma_h = \sigma_V$; Heidback 2014, verbal communication). Observations in the URL show that for tunnels oriented parallel to the maximum horizontal stress (σ_H) shear fractures are associated with inward block movement in the pillars. For openings parallel to the minimum horizontal stress (σ_h) fractures in the roof and invert, dip toward and away from the face, respectively. The favored orientation for stability of HLW borings is therefore parallel to σ_H. All casings and liners need to be backfilled to accommodate anisotropic deformation.

Creep is defined as time dependent deformation with no change in pore pressure and no change in mean stress. Similar long-term deformation rates are observed at Mol, Bure, and Mont Terri suggesting a common mechanism (Armand 2014, verbal communication). Data from Mont Terri *in situ* tests suggests this mechanism could be gradual pore pressure dissipation.

3.3.2 St. Martin La Porte Access Adit

The Lyon-Turin Base Tunnel is a direct rail link crossing the southern Alps between France and Italy. When completed, the main tunnels will be approximately 50 km long, with a maximum overburden depth of 2,500 m. Three adits are being driven to intersect the tunnel alignment, for access during construction and for services after the rail tunnel is put into operation. The Saint Martin La Porte access adit was begun in 2003 and completed in 2010, at a final length of 2.3 km. This adit project has provided important information on tunneling conditions, which has been applied for the main tunnels. It is included here because the adit traverses a complex sedimentary geologic section where severe squeezing conditions were encountered, for which novel support methods were developed (Bonini and Barla 2012).

Construction started using stiff supports that included steel ribs and heavy shotcrete, which became overstressed and failed by cracking and buckling. Steel ribs with sliding joints were not effective because of uneven deformation. After about 1.5 km of tunneling cracking of the shotcrete, failure of the steel ribs, overbreak, and large convergence (up to 2 m) construction was stopped and a new staged, yielding support system was introduced (Bonini and Barla 2012). This system excavated the tunnel in three stages: 1) installation of grouted fiberglass rods around the tunnel perimeter ahead of the face; 2) mechanical excavation 1 m at a time, with installation of grouted bolts, yielding steel ribs, and 10 cm of shotcrtete; and 3) about 30 m back from the face, the tunnel was widened to full size, with support consisting of yielding steel ribs and 20 cm of reinforced shotcrete, and slots cut in the shotcrete and filled with deformable elements (plastic foam aggregate concrete). Up to 60 cm of tunnel closure was accommodated by the initial support, and up to 40 cm after the final support installation.

The feature of this system most important for constructing the Lyon-Turin Base Tunnel and other tunnels, is the control of deformation and stress in the lining using deformable elements (Bonini and Barla 2012). This experience may be regarded as a successful solution to ground conditions that might be "worst-case" and experienced only locally during construction of a repository.

3.3.3 Channel Tunnel

The Channel Tunnel is 50 km long, connecting France and Britain with two rail tunnels and one service tunnel. It is included here because it is famous, and because it serves as a benchmark for tunneling methods and (high) cost. The maximum water depth in the English Channel along the tunnel alignment is 55 m, and the maximum depth of the tunnel is an additional 45 m below the seabed. The rail tunnels are 7.6 m in diameter (finished) and the service tunnel is 4.8 m in diameter (finished). Tunnels are lined with pre-fabricated segments of cast iron or high-strength concrete (Robbins 2014d).

The British-side rock is self-supporting chalk marl, homogeneous, with low-permeability (Gibson et al. 1989). TBM features were selected to accommodate possible "blocky ground" or high water inflow, with double-shields and an erector for single-pass segmented liner installation. The shields were for weaker ground conditions. To address the possibility of high water inflow seals were included, integral to the shields, and waste rock handling was made more flexible with a retractable conveyor and sealable opening in the pressure bulkhead. Expandable "tail shutters" were included to seal against the installed liner if necessary. The thrust system and TBM structure were designed for loads that might occur with pressurization. The French-side rock is also chalk marl, but highly fractured, with high water inflow considered more likely, fed by connections to the sea above. EPB TBMs were designed to operated under high water pressure or dry conditions (Robbins 2014d). They were operated in EPB mode for the first 5 km, then as open-type TBMs to completion.

Tunneling was completed over a period of approximately four years, ending in 1991. After completion, the TBMs from the British side were driven steeply downward and abandoned, while the French-side TBMs were disassembled and removed. Total cost for the project including lining and track, two underground crossover stations, transfer facilities at each end, and rolling stock was approximately $7B, representing an 80% cost overrun (1988 dollars; escalation for 26 years at 3% per year is approximately 220%) (Flyvbjerg 2005; Wallis 1993). This figure does not include financing and corporate costs which put some estimates at more than $20B (1996 dollars).

3.4 United States

For many years tunneling projects in the U.S. were dominated by the U.S. Bureau of Reclamation and the U.S. Army Corps of Engineers, performed for flood control, hydroelectric power, and water conveyance. Several examples are discussed below. More recently, extensive tunneling has been performed to control environmental damage from combined-sewer-overflow (CSO), and the largest two of these projects are also discussed. Some of these tunnels are shallow compared to a geologic repository, for example the penstock tunnels at Oahe, and the CSO tunnels at Chicago and Cleveland. The value of these examples is that they provide cost benchmarks and engineering solutions similar to what could be implemented for repository construction (projects discussed in this report are summarized in Table 1).

3.4.1 San Juan-Chama Project – New Mexico

The San Juan–Chama Project (SJCP) is a series of channels, tunnels, diversion dams, storage dams, and reservoirs that conveys water from the San Juan River in southern Colorado, across the Continental Divide to Azotea Creek, and eventually to the Rio Grande River in northern New Mexico. Water is thereby diverted from the Colorado River system to the Rio Grande. The project was constructed by the U.S. Bureau of Reclamation in the 1960s and 1970s, and now provides much of the municipal water for the city of Albuquerque and smaller communities upstream along the Rio Grande River.

The SJCP features of interest for this report are three water conveyance tunnels: the Blanco, Azotea, and Oso tunnels (Table 3). Each of these is circular in cross-section and lined with unreinforced concrete (USBR 1989). Test borings and geophysical surveys (surface and airborne) were performed during design. During construction, borehole grouting was often used for stabilization and to control groundwater inflow during construction. The SJCP tunnels were excavated mostly with TBMs, with drill-and-blast methods used for setup and contingencies (USBR 1989). Unconsolidated glacial deposits were difficult to mine through because of instability and water inflow, but were mitigated with steel ribs and timber lagging (Glaser 2010). Advance rates averaged roughly 1,000 meters per month, depending on rock type (Cannon 1967). Methane intrusion at potentially explosive concentration (1%) was encountered in the Azotea and Blanco tunnels.

Tunnel construction cost averaged less than $1,000 per lineal meter, fully lined (1989 dollars; escalation for 40 years at 3% per year is approximately 350%). Importantly, these tunnels have already provided more than 40 years of service to a critical water supply mission, with little or no maintenance (Atwater 2014, personal communication). The low maintenance needs may be partly because parts of the SJCP tunnel system are above the water table so that swelling due to moisture intrusion could be avoided. However, cover depth exceeds typical water table depth in many tunnel sections, and significant water inflow (80 to 400 liters per minute) was observed in all three tunnels. Clearly, stability of the tunnel linings was achieved by stabilizing moisture conditions in the shale, which can exhibit swelling behavior.

Table 3. Summary data for San Juan–Chama Project tunnels.

SJCP Tunnel	Excavated Dia. (m)	Finished Dia. (m)	Length (km)	Max. Depth (m)	Complete (Cost 1989$)	Geology/Remarks (USBR 1989)
Azotea	4.3	3.3	20.6	488	1964-70 ($17.1M)	Mancos and Lewis Shales, sandstone, siltstone, glacial debris; numerous faults; steel ribs over 36% of length
Blanco	3.1	2.6	13.9	600	1965-69 ($9.8M)	Lewis Shale, sand, gravel; squeezing encountered; rock bolts; liner plate over 9% of length
Oso	3.1	2.6	8.1	232	1966-70 ($5.8M)	Shale, glacial debris, clay; ~100 m^3 ravelling encountered; rock bolts; steel ribs over 3% of length

3.4.2 Phase 1 TARP – Chicago

The Chicago Tunnel and Reservoir Plan (TARP) Phase 1 project is the largest ever constructed for managing CSO. Rainfall runoff on streets and highways is directed to sanitary sewers, producing flow that far exceeds the capacity of water treatment facilities. This CSO is directed to TARP drop shafts, connecting to extensive tunnels, for storage. Eventually the CSO is pumped out to surface reservoirs which are still under development as part of TARP Phase 2, then treated and released to surface waterways.

The TARP Phase 1 project comprises four tunnel systems: Mainstream, Des Plaines, Calumet and O'Hare, which have a combined length of 176 km (AUCA 1999). The tunnels were built to store combined sewer overflow (CSO) during rainfall events, to be pumped out later and reclaimed for discharge to the environment. Typical tunnel diameter is 10.8 m, and depth ranges from approximately 50 to 100 m (actual depths vary for each tunnel). Although the TARP tunnels are very likely shallower than a repository would be, and in carbonate rather than argillaceous rock, they are included here because the scale of TARP project construction is similar to that which would be associated with a repository in physical dimensions, duration, and cost.

All the TARP tunnels are constructed in gently dipping Silurian dolomite, which is homogeneous and slightly argillaceous. The dolomite compressive strength ranges from 95 to 125 MPa, which is relatively high and explains why hard-rock TBM designs were used. The host rock has two widely spaced joint sets, and rock quality (RQD) ranging from 91 to 98% (AUCA 1999). Groundwater inflow was mainly through joints and bedding planes, but is now controlled by the installed liner. Occassional shear zones produced inflows on the order of 400 l/min.

Approximately 30 TBMs were used over the life of the TARP Phase 1 project, as the four tunnel systems were each mined in multiple segments, sometimes simultaneously, and side-tunnels connecting to drop shafts for runoff were mined at smaller diameters. The TBMs used were typically of the single-shield open design, with retractable grippers. Immediate ground support generally consisted of rock bolts, followed by a cast-in-place liner of 27 MPa (4,000 psi) unreinforced concrete, nominally 0.3 m thick (Balogh 1996). Some parts of the facilities were reinforced with steel, and/or used concrete of higher strength, as needed for the CSO intake, storage and pumping facilities.

Construction of the TARP tunnels began in 1976 on the Mainstream Tunnel, and finished in 2006 with completion of the Little Calumet Tunnel. Approximately $4B has been spent to-date for 176 km of underground construction, including connector tunnels, shafts, intake structures, etc. ($23k/meter average cost). There has been some repair and rework of TARP tunnel sections, but the project life is expected to be unlimited (EPA 1988).

3.4.3 Euclid Creek Project – Cleveland

The Euclid Creek Storage Tunnel (ECT) is a CSO storage tunnel designed to protect Euclid Creek and Lake Erie from a substantial fraction of overflow release events each year. The project customer is the Northeast Ohio Regional Sewer District. The ECT runs along Lakeshore Boulevard and the Lake Erie waterfront in Cleveland. A 160 Mgal/day pumping station will discharge stored wastewater between storm events, to an existing treatment plant. The ECT has a 7.2-m (finished) diameter and will be 5.5 km in length. A similar effort is underway in Indianapolis, IN with the Deep Rock Connector Project, a 6-m diameter, 12.8 km long concrete lined tunnel, in limestone and dolomite at a depth of approximately 75 m under the city, that will connect a 40-km system of tunnels (Van Hampton 2013).

The ECT was excavated in the Devonian Chagrin Shale member of the Ohio Shale formation, at depth ranging from 57 to 66 m (HMM 2010). This shale is horizontally bedded, blue-gray to dark-gray, weak to medium strength, with thin interbeds of shaly, calcareous siltstone and sandstone. Approximately 80% of the test core had an RQD of 80% or better. Unconfined compressive strength of core samples averaged approximately 65 MPa, making this a robust, indurated lithology compared to claystones, mudstones, and soft shales being considered for repository development. The slightly weathered to unweathered Chagrin Shale has significant slaking (swelling) potential. Northeastern Ohio is known for high horizontal stresses, and a horizontal:vertical stress ratio of 2.0 to 2.5 was observed and applied in stability analysis. The Chagrin Shale is known for “horizontal slabbing” behavior in excavations (Robbins 2014a). However, over-stress conditions (exceeding compressive strength) did not occur in the ECT, probably mostly because of the shallow depth. (Such over-stress conditions could be expected at repository depths of 300 m or greater, which could require changes in the excavation method and more robust ground support.)

A total of 60 test borings were drilled along the ECT alignment and at locations of key facilities such as shafts. The shale is overlain by as much as 30 m of glacial soils along the tunnel alignment. Auger probe holes were used to test the depth to shale bedrock, and natural gas (CO_2, H_2S and flammable alkanes) was observed.

The ECT project is expected to be complete in 2015. Tunnel boring and shaft construction were completed in November, 2013. Tunneling was performed with a Herrenknecht double-shield hard rock TBM, with disc cutters, to a rough diameter of 8.2 m. An Alpine roadheader mining machine was used for the 37-m TBM launch chamber and the 90-m tail tunnel. Waste rock was removed through a 12-m (finished) diameter mining shaft. The one-pass tunnel liner consists of pre-fabricated segments made from steel-fiber reinforced concrete. Liner grouting was performed through the tail of the TBM using a fast-setting two-component grout (TBM 2013). The fast-setting grout quickly established a barrier to natural gas inflow, controlled swelling, and also limited grout intrusion into the TBM area forward of the liner. The one-pass liner design accelerated the schedule (compared to the predecessor Mill Creek Tunnel Project discussed below). A total of five shafts were constructed, ranging in size from 4.9 m to 15 m in diameter, and four will have baffle structures to convey stormflows into the tunnel. Total project cost is estimated to be approximately $200M (current dollars), or about $35k per meter of completed tunnel, or $800 per cubic meter of wastewater storage capacity. This total cost includes lining, shafts, and intake structures (but not the pumping station).

3.4.4 Mill Creek Tunnels - Cleveland

The Mill Creek Tunnels, Phases 1, 2 and 3 were completed prior to the ECT, for CSO storage. The Phase-1 tunnel is smaller: 3.8 m excavated diameter and 4.1 km long, completed in 2002 at a cost of $23M (1997 dollars; $6k per meter).

The Phase 2 and Phase 3 tunnels are more closely comparable to the ECT. The Phase 2 tunnel has a finished diameter of 6 m and is 4 km long. It was also constructed in the horizontally bedded Chagrin Shale, at depth ranging from 49 to 79 m, and a final 1999 cost of $57M ($14k per meter). The Phase 3 tunnel is similar, again constructed in the Chagrin Shale, with a length of 4.5 km and depth ranging from 63 to 93 m. Both the Phase 2 and 3 tunnels were constructed with a double-shield Robbins TBM, with disc cutters. Ground support consisted of steel ribs with steel mat lagging and some timber lagging where excessive rock breakage (i.e., overbreak) occurred in the tunnel crown. Overbreak of up to about 0.65 m was attributed to loosening and over-stress fracturing, mostly in the Phase 3 tunnel. It occurred over about 55% of the tunnel length (HMM 2010). Stresses in the rock around the openings were generally much less than the compressive strength discussed previously, suggesting that bedding separation or "slabbing" occurred during excavation. Excavation was completed in December, 2008 with typical advance rate of 3 m per hour. Final construction included a cast-in-place, unreinforced concrete liner with minimum thickness 0.3 m, using 34-MPa (5,000 psi) concrete (Figure 9). The original construction contract for $58M was overrun numerous times resulting in a final 2002 cost of $85M ($19k per meter). Graft was a factor resulting in three criminal convictions and a settlement of $12M to the project owner (Plain Dealer 2014).

3.4.5 Pierre Shale

The Pierre Shale was considered for possible repository development in the 1970's and 1980's (Gonzales and Johnson 1984; Hansen et al. 2010). The Pierre and correlative Cretaceous facies (e.g., Lewis Shale, Mancos Shale) are well known and widespread across the northern Great Plains and Rocky Mountain regions. The Pierre Shale is thick, "reasonably uniform" (Shurr 1977), highly deformable, with low permeability (Tourtelot 1962; Neuzil 2013, 1986). It is classified as a mudstone, with abundant clay mineral content, and lower strength than the Opalinus Clay or COX argillite (see summary table of Hansen et al. 2010).

Like other fine-grained clay-rich rock types it would likely exhibit viscoplastic response (i.e., creep) to repository loading conditions (Nopola 2013). It is included here because of ongoing interest (Roggenthen et al. 2013) and because it represents the challenges that would likely be associated with large-scale underground construction in a soft, poorly indurated smectite-rich rock. This discussion could apply also to other strata such as the Lewis and Mancos Formations.

Early mines excavated in the Pierre Shale, typically in the process of developing other geologic features or resources (e.g., Pettit 1946). The Harold D. Roberts Tunnel in the Colorado Front Range, passes through the Pierre Shale in a geologically complex, alpine setting (Wahlstrom et al. 1981). These examples are generally not applicable to the Pierre Shale in the Great Plains because of geologic conditions and excavation methods.

Figure 9. Mill Creek Tunnel Phase 2 (Cleveland, OH) showing steel ribs and lagging support (upper) and final cast-in-place concrete liner (lower) (with permission, HMM 2010).

There are few TBM excavations in the Pierre Shale, and the most reknown are the penstock and outlet tunnels of the Oahe Dam in north-central South Dakota. Excavation began in 1955 using a first-generation TBM (Robbins 2012). The dam has seven steel penstocks 7.3 m in diameter, 1,000 to 1,221 m long, which are embedded in tunnels mined in the Pierre Shale. Another six tubes for release of high flows are 6.0 m in diameter, 1,066 to 1,116 m long, and similarly embedded (ACoE 2014). Thus, there are approximately 14 km of large-diameter, TBM-excavated tunnels at the Oahe Dam. The project also marked the first large-scale use of pre-fabricated reinforced concrete liner segments. The penstock and outlet works tunnels at Oahe were mined at grades of a few percent, starting at intake structures constructed in the reservoir and passing underneath the dam abutments. These are early examples of TBM-constructed ramps such as that which could be built for waste handling at a repository.

Nopola (2013) performed ground support performance calculations for a circular tunnel in Pierre Shale, at depth of 300, 500 or 700 m. The rock was assigned viscoplastic creep properties estimated by analogy to published test data for the COX argillite and the Kanawha Formation siltstone and shale. Concrete liner properties (e.g., compressive strength 60 MPa) and thickness (0.25, 0.5 and 0.75 m) were selected to bear the load transferred over 20 years, from creep deformation of the host rock (Figures 5 and 6). Finished tunnel diameter was assumed to be 3 m. *In situ* stress conditions were assumed to be lithostatic. In the simulations, the liner was emplaced after elastic deformations had occurred in response to excavation. These results are a useful starting point for further design analyses and site specific evaluations for the Pierre or other similar shale units.

3.4.6 Other Large-Scale Tunnels in Argillaceous Rock in North America

Flathead Tunnel, Montana – This single-track rail tunnel was constructed between 1966 and 1969, in quartzite and argillite, generally medium to hard (Skinner 1974). The tunnel is 11.3 km long, constructed by drill-and-blast methods, with a horseshoe cross-section. It remains the second-longest rail or highway tunnel in the U.S. (first is the Cascade Tunnel). It cost $37.9M to construct, inclusive of lining, track, utilities and ventilation ($3,363 per meter; 1969 dollars).

Niagara Tunnel #3, Ontario – This hydroelectric tunnel was bored under the city of Niagara Falls, Ontario, and completed in 2013 (OPG/Strabag 2014). It runs parallel to twin 13.7-m diameter (finished) tunnels completed in 1955, that divert flow from the Niagara River, around the Niagara Falls, to a hydroelectric power station downstream. Tunnel #3 has a 12.5-m finished diameter (14.4 m excavated), and is 10.4 km long, with a maximum depth of 140 m. Much of the tunnel was excavated in the Ordovician Queenston Shale (mudstone facies) which exhibited fracturing and roof breakout throughout the excavation. An open-type TBM was used, of a single-shield hard-rock design with grippers, possibly the largest TBM ever built and used at the time. Initial ground support consisted of rock bolts, wire fabric, and shotcrete. An impervious membrane was then installed, and the final cast-in-place concrete liner. Because this is a pressure tunnel, the entire liner assemblage was pre-stressed by high-pressure grouting. Tunnel #3 was designed for a life of at least 90 years, and cost approximately $1.5B (2009 dollars) inclusive.

Boston Harbor Project – As part of the project, a wastewater outfall tunnel was constructed from Deer Island to Massachusetts Bay, beneath Boston Harbor, constructed from 1992 to 1996 (Robbins 2014b). The tunnel diameter is 8.0 m (finished), and it is 15.2 km long. The predominant rock type is Cambridge Argillite in beds 1 mm to 8 cm thick, also volcanic flows, tuffs, igneous dikes and sills, etc. A double-shield TBM was selected to handle variable rock conditions and water inflow. A pre-fabricated segmented concrete liner was installed during excavation. The multiple TBM operation modes permitted pushing against the liner, or grippers, for advance depending on rock conditions. Borehole grouting was used extensively where hard, fractured rock and high water inflow were encountered. Total cost of the tunnel and associated outfall management facilities was $3.4B, or approximately $224k per meter (Holmstrom 1999).

Park River Tunnel, Connecticut – This water conveyance tunnel was completed in 1979 to transfer a portion of the Park River flow to the Connecticut River (Bieniawski 1990). It has a diameter of 6.6 m (finished), is 2.8 km long, and has a maximum depth of 60 m. The geology consisted of dipping Triassic sand red shales and siltstones, interrupted by basalt flows. Fractured rock and fault zones were also encountered. A TBM was used for the excavation and installation of a pre-fabricated segmented concrete liner 0.23 m thick. Ground support was increased in fractured rock, or where close to the surface. Steel rib supports were used in fault zones. The segmented liner was pressure-grouted to pre-stress it for service. The tunnel was completed at the bid price of $23.3M ($8.3k per meter, 1978 dollars).

Plateau Creek, Colorado – This water conveyance project involved 21 km of pipelines and tunnels, including two tunnel sections totaling 4.1 km. The tunnels were 3.3 m in diameter (excavated). They were constructed in well-indurated sandsone, shale and siltstone, using an open-type, hard-rock TBM (Robbins 2014c). Tunneling was started mid-2000 and completed in March, 2001. Ground support consisted of rock bolts, wire fabric and shotcrete. The tunnels were completed at the contract cost of $14.1M ($3.4k per meter; Tunnelbuilder 2014).

4. Discussion, Summary and Repository Construction Scenario

The 50- to 100-year opening stability required for many repository openings can be achieved, by analogy to rail, highway, water conveyance and hydroelectric tunnels in clay/shale media. Selection of self-supporting (at the disposal development depth) and low-permeability host rock will significantly lower costs by allowing use of open-type TBMs with dry operations. Creep response may impact cost also, since tunnels in more viscoplastic media like the Pierre Shale may require heavier ground support than less creep-prone media like the Opalinus Clay and COX argillite. A shallower repository could significantly reduce excavation and construction costs in clay/shale media, and improve long-term stability (e.g., 200 to 300 m depth, instead of 500 m).

Fastest construction is achieved using TBMs with a single-pass, pre-fabricated, segmented concrete liner. Liner segment properties (thickness, strength, reinforcement) can be adjusted to provide needed strength for a range of conditions. Such a pre-fabricated liner can be backfilled with grout after installation to improve mechanical coupling and minimize groundwater inflow wherever it might occur. Use of compliant materials for backfilling such as pea gravel (uncemented) or concrete with plastic-foam aggregate, has proven useful to allow large deformations of the host rock and control liner stress. For host media containing clay minerals and possibly overconsolidated (previously deeper burial), the liner may need to be emplaced and sealed immediately to prevent moisture intrusion and swelling (this will depend on the availability of moisture in the newly constructed tunnel opening).

Cost data only for excavation and ground support are difficult to obtain. Published cost data typically lump together tunnels with a wide range of other project features such as portals, shafts, pump houses, underground stations, pressure chambers, rail or roadbed, finance costs, etc. Also, cost data presented here are for tunnels that range from 3 m to 17 m in diameter, and that were completed over a period of approximately 50 years so they must be escalated for comparison. Nevertheless, the summary in Table 1 suggests that construction cost on the order of $10,000 per meter or less may be possible for most repository drifts. For a large repository with 300 km of drifts, this equates to a total tunneling/lining cost of less than $3B. The wide range of cost data, even considering excavated volume and ancillary facilities, suggests that experienced management is essential for controlling costs.

Repository Excavation/Construction Scenario in Clay/Shale Media – This scenario describes how a repository could be constructed for disposal of large waste packages containing up to 17 MT of spent fuel (equivalent to 37 pressurized water reactor fuel assemblies).

In-drift disposal is used (Hardin et al. 2013) with packages placed on low pedestals or directly on the floor, approximately 30 m apart (center-center). Emplacement drifts are parallel and arranged in panels for access and to control ventilation. Cementitious materials are used extensively in construction and have been thoroughly characterized so the possible impacts on longevity of the waste form and packaging, and on radionuclide transport in the host medium, are well understood.

A thick, soft, flat-lying clay/shale host formation is selected that extends to a depth of 500 m (the Pierre Shale is a close example presented in this report). The repository horizon is identified and characterized at a depth of 300 m. A ramp is then constructed from the surface to repository depth, to be used for initial access and eventually for waste transport underground. A TBM is selected based on rock properties, *in situ* stress and hydrogeologic conditions. Rubber-tire equipment can operate at grades up to 10% (Fairhurst 2012) but for TBM excavation and to mitigate operational hazards, a grade of 5% or less is used, resulting in a ramp at least 6 km long. The ramp diameter (and that of the main drifts described next) is approximately 8 m (finished). Permanent ground support is installed consisting of pre-fabricated, reinforced concrete segments. Larger, heavier invert segments with extra reinforcement serve as the ramp running surface. The liner and invert are fully grouted to prevent groundwater inflow or swelling, to stabilize the liner, and to transfer running loads from the invert to the host rock.

Main drifts at the repository horizon are then constructed using the same TBM, liner and invert specifications. The drift layout will support underground testing activities for characterization, repository design, and licensing. It will then support initial construction of repository emplacement drifts. A simple drift layout is used that will later facilitate construction of disposal panels. This layout is large enough to tie together the minimum number of shafts that will be needed for repository construction (men-and-materials, ventilation, etc.). These shafts are constructed by raise-boring and lined with concrete, as the main drifts are being excavated. More shafts may be constructed later to service additional disposal areas further away.

Once the underground infrastructure is established and repository construction can begin, another, smaller TBM with a diameter of 5 m or less is assembled underground. This TBM will excavate and install ground support in the emplacement drifts. The diameter is reduced to control the amount of backfill needed at repository closure, and the peak backfill temperature. Ground support consists of a pre-fabricated, segmented, backfilled, concrete liner. The quantity of ground support material (e.g., liner thickness) is controlled to limit the impact on the disposal environment after permanent closure (while meeting long-term preclosure stability requirements). Before waste emplacement, operable radiation shielding and ventilation regulators are installed. Emplacement drift seals and plugs are pre-constructed to facilitiate closure (these interrupt possible future flow paths along the liner or in the host rock excavation-damage zone).

At closure, all the repository drifts are backfilled with granular, swelling clay-based material in a dehydrated form. The functions of the backfill are to prevent large-scale movement of groundwater along the repository openings, and to provide mechanical support when the tunnels eventually collapse. Backfilling is done remotely in the emplacement drifts, and is a process that will have been analyzed and tested to support the original licensing process. Construction of plugs and seals is completed, and the repository is closed. Monitoring continues as long as needed to ensure that system performance is safe and complies with licensing requirements.

5. References

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Appendix A – Trip Report: Symposium on Rock Mechanics and Rock Engineering of Geological Repositories in Opalinus Clay and Similar Claystones

This trip report was filed by the author after attending the symposium on February 14, 2014, in Zurich, Switzerland. Presentations and additional information are available at http://www.egt-schweiz.ch/index.php?id=symposium&L=2. This 1-day symposium was principally organized by Prof. Dr. Simon Löw of the Technical University – Zurich (ETH). The stated purpose was to help the Swiss repository implementing agency (Nagra) determine if enough is known about claystone mechanics for it to recommend that the siting process proceed past Stage 2 (a process defined in the established Swiss siting protocol). There was extensive discussion of rock conditions at the Mont Terri URL, which is situated roughly 300 m below the surface (depth of potential repository sites could be 800 m or more).

The symposium was conducted in three parts: 1) laboratory and *in situ* experiments, and models; 2) underground construction experience; and 3) proposed repository layouts and construction methods. The first part was presented by professors from academia, while underground construction experience was presented by investigators from Mont Terri and by Swiss tunneling engineers, and repository design information was discussed by staff from government agencies and R&D institutions. All presentations described below were specific to the Opalinus clay unless noted otherwise.

A.1 Laboratory and In Situ Experiments, and Models

Florian Amman (ETH – Zurich) described the rock mechanics challenges as anisotropy, sensitivity to moisture, describing brittle and nonlinear behaviors, time dependent deformations (consolidation, creep, swelling), and the effects of suction. In soil mechanics the solids are typically considered to be incompressible, while in clay/shale media they must be considered compressible. Simple mechanical tests (e.g., unconfined compressive strength, UCS) are not so simple when saturation is required, to represent *in situ* conditions and to produce consistent and comparable data. The soils testing literature contains useful guidelines for saturation methods and time required. Effective formation properties are calculated from undrained tests at *in situ* saturation. Note that Opalinus strength is greatest when suction is approximately -70 MPa which corresponds to a liquid saturation of roughly 50% or less. Describing transversely isotropic elasticity with pore pressure coupling requires seven constants including Skempton parameters that couple pore pressure with normal stress parallel and perpendicular.

Heinz Konietzky (Technical University – Bergakademie Freiburg) described modeling of rock structures using Voronoi bodies (UDEC code from Itasca Consulting Group) or rigid spheres (PFC code from Itasca). A variety of bulk deformation and fracture mechanics mechanism can be represented. The Opalinus was described as particles of clay, silt and carbonate, adhered by constitutive particle bonds.

The Mont Terri excavation sequence, confirmed by experimental measurements, consists of excavation, unloading, pore pressure decrease near the opening for several months, leading to changes in loading. Both tensile and shear cracks are observed at the drift wall. Pore pressure increase may be steep during early time after excavation.

Silvio Giger (Nagra) stated that using size-based geotechnical classification, the Opalinus would have a low clay fraction (<20%) and would be a siltstone. Using mineralogy the Opalinus is 40% to 70% clay, with the remainder quartz/feldspar, and carbonate. Porosity of the Opalinus may not be correlated to current burial depth. The UCS is 10 to 25 MPa perpendicular to bedding, and 4 to 7 MPa at 30° to bedding. Tensile strength decreases as saturation increases (consistent with the effect of suction). The protocol for UCS testing calls for undrained, with up to 2 weeks consolidation under pressure before testing. The compressive strength testing strain rate is 10^{-6} sec^{-1}. Creep or liquefaction testing is done with deviatoric stress up to 60 MPa and mean stress up to 30 MPa.

Frederic Pellet (University of Lyon, France) described microstructural and petrophysical evaluations to predict dilation and cracking in shales, which may be time-dependent, forming over 100's of years. He claimed that clay/shale behavior needs to modeled as creep, where the strain rate tensor varies as the ratio of deviatoric to equivalent stress. Implementation of creep flow potential using the von Mises definition of equivalent stress (and by analogy, the Tresca definition) is generally suited for isotropic media, and a more complete implementation is needed for transversely isotropic media. If creep is important then the EDZ could continue to evolve as the rock flows and loads are transferred further away, or into the liner. Fracture closure and sealing in the near-field host rock might not occur. The COX argillite has been tested at 10^{-6} and 10^{-8} sec^{-1} strain rates. Transgranular cracks were observed at higher strain rates, intergranular cracks at lower ones. The fundamental mechanism of plastic deformation and cracking appears to be dislocation. For the COX argillite at Bure, the UCS is 27 to 29 MPa, Young's modulus is 5 to 6 GPa, and porosity is 15.5%.

For the argillite at Tournemiere, UCS is 35 to 38 MPa, Young's modulus is 21 to 27 GPa, and porosity is 9%. Desiccation cracks are prevalent on all vertical surfaces exposed to air, with spacing of ~20 cm. In the winter these cracks open to apertures of ~2 mm, while in summer they close fully. Horizontal and vertical cracks behave similarly. Displacements are observed for relative Humidity (RH) fluctuations greater than ~15% and periods greater than ~6 hr. These displacements appear to be reversible.

Finally, Pellet instigated a spirited discussion by claiming that because there is "almost no free water" in argillites, that there is "almost no pore pressure" so that the effective stress concept does not apply and coupling to framework stress is not important. He said that Biot poroelasticity is not applicable because there are "few Hertzian contacts" (Hertzian contacts can be stiff). Also, he said that suction would effectively be minimal because of "cavitation" (ubiquitous voids).

A.2 Underground Construction Experience

Derek Martin (Univ. of Alberta) lectured on rock mechanics in claystones and similar media. Clay/shale media can be classified on UCS (0.3 to 30 MPa following the ISRM classification scheme). They are often overconsolidated reflecting previously greater burial depth. Strength is a function of water content, and these media have little or no true cohesion so the effect of water retention (suction) matters. Stress drivers include $\frac{\sigma_1}{UCS}$ and $(\sigma_1 - \sigma_3)$.

The bulk rock exhibits some swelling on unloading, and squeezing if over-stressed. At Mont Terri these responses were investigated in the ED-B test and the Mine-By Experiment (the second at the site). Convergence (diameter closure) was found to be 0.1% to 0.2% in the ED-B, and 1% to 1.5% in the Mine-By. The former test was driven perpendicular to the strike of bedding (which dips at ~40°), and latter was parallel. (For comparison, convergence in the Boom Clay is >2.5%, and 1% to 2% at Bure.) Slip on bedding was not observed at Mont Terri. Pore pressure increased ~1 MPa at the advancing face, and decreased 2 to 3 MPa where unloaded. Pore pressure effects were observed at up to 3 diameters from the drift centerline.

Eduardo Alonso (Technical University of Catalonia) discussed the effects from transient deformation (not creep) on permeability. He referred to the HG-A test at Mont Terri, and the associated reports, for information on permeability of the EDZ. Most flow occurs along fractures or other preferential pathways, so permeability depends on the evolution of discontinuities, dilation, etc. The Barcelona Basic Model (BBM) is a reasonable constitutive model for the matrix, with addition of anisotropy. Ubiquitous fractures can be used to represent discontinuities that open due to excavation. Fracture permeability is represented using the cubic law, while unsaturated properties can be represented using a Leverett scaling approach. Initial porosity is heterogeneous in model grids, and is assigned randomly. Unfortunately, the constitutive model with these features will have more than 20 parameters. Any state-of-the-art thermal-hydrological-mechanical (THM) code can be used, but Code_Bright has generated some reasonable agreement with field test (HG-A) results. Permeability develops along stress paths, or regions of the rock mass that are similarly stressed. Important uncertainties remain, for example, the density of discontinuities and their properties. The aperture-porosity and aperture-permeability relationships can increase numerical effort, and more linear formulations would be helpful.

Next Paul Bossart (Swisstopo, Mont Terri Project Director) described the excavation, construction, and ground support engineering experiences at Mont Terri. Excavation methods that have been tried include: drill-and-blast, roadheader, hydraulic hammer, raise boring, and steel-toothed auger. The drill-and-blast method produced the greatest overbreak and extent of excavation damage. The roadheader is the method of choice because of low cost, no water needed, limited spalling or overbreak, limited EDZ, and workable dust control. For dust control, two suction fans are used with filters.

The Mont Terri tunnels are designed for R&D, with a lifetime on the order of 20 years, with only local repairs needed. Immediately after excavation a thin layer (5 cm) of shotcrete is applied to prevent hydration or slaking. All shotcrete is formulated for early strength and low pH (balanced portlandite, excess soluble silica, and super-plasticizer). Wire mesh (e.g., coarse “welded wire fabric”) is then installed with short bolts. Long rock bolts may then be installed for larger openings, or where dictated by performance requirements (e.g., minimize maintenance, maximize service life). Rock bolt length is typically 1× to 1.5× opening diameter, and they are installed using a full 270° pattern. Bolts may be metal or fiberglass, and may use point anchors or full grouting. Alternatively, steel sets may be installed with “distortion zones” at the springlines.

After a delay of several months to allow convergence, a final layer of shotcrete is applied (15 cm). As much as 5% convergence has been observed where stress is concentrated by nearby excavations (e.g., the alcove from which the FE test drift was constructed;). The floor is shotcreted all the way to the ribs, and the surface is worked to provide a running surface for vehicles and water flow. Drift convergence will continue until the floor is shotcreted. At Mont Terri, a few percent of the total drift length has been impacted by shotcrete slabbing on the ribs, which occurs in these locations with a frequency of 10 to 20 years. Renovation consists of removal of loose shotcrete and rock using a hydraulic hammer, replacement of wire mesh, and application of new shotcrete.

Rock damage at Mont Terri is dominated by bedding-induced breakouts. Stress induced breakouts are uncommon (at 250 to 400 m burial depth). Some of the drifts aligned parallel to the strike of bedding have experienced shear zone failure, with large-block (e.g., 1 to 2 m thick) movements detected using borehole extensometers. The EDZ at Mont Terri typically exhibits "plumose hackles" (Martin and Lanyon 2001) and the mechanism may be related to pore pressure excursions. Hydraulic conductivity of the EDZ at Mont Terri is on the order of 10^{-8} m/sec, and gradual reduction is observed ("sealing"). For a deeper repository in the Opalinus (e.g., up to 800 m) the intensity and spatial extent of the EDZ would be significantly greater.

Walter Steiner (B+S AG, a tunnel engineering firm) described tunneling in the Swiss Jura. Early-modern tunneling methods in Switzerland were distinguished by use of movable excavation shields to stabilize rock before liner installation. The earliest methods used drill-and-blast with a shield, and a segmented liner (e.g., pre-cast concrete or cast-iron). For example, the Baregg tunnel (A1 motorway, tubes 1 and 2) was constructed by drill-and-blast with a horseshoe-shaped shield, and completed in 1970. Non-circular shields were found to be unworkable because they have a tendency to roll and cannot be readily corrected or steered. In 1970 the Heitersberg rail tunnel was constructed using a circular, open Robbins TBM (10.8 m diameter). Ground support consisted of an outer liner of shotcrete sprayed by a robot attached to the TBM, with wire mesh and rock bolts, and steel supports where needed (24%). Completed in 1985, the Gubrist highway tunnel (tubes 1 and 2) was excavated using a similar arrangement and the same shield, and the liner was mated with the shield to improve the stability of the interval between them. Another early highway tunnel, the Rosenberg tunnel, was excavated using a shield with four road headers.

Soft-rock tunneling in Switzerland advanced after the 1970's, accelerated by extensive tunnel construction for the A16 and other motorway routes. The Mont Russelin highway tunnel completed in 1998 is 3.5 km long, with more than 300 m of maximum overburden. The Bözberg twin tubes (A3 motorway) were completed in 1994, and are 4.3 km long with more than 200 m maximum overburden. The Adler rail tunnel was completed in 2000, is 5.2 km long, and was excavated to a diameter of 12.5 m (then a record). The Bure highway tunnel was completed in 2011, is 3.1 km long, and has a diameter of 12.6 m. The Mont Terri highway tunnel was completed in 1998, is 4 km long, and replaced the tunnel now used to access the URL.

Tunneling Opalinus Clay has achieved best results with single-shield TBMs, avoiding the problems associated with conventional methods such as drill-and-blast. The tunnel should have circular cross section, and a continuous segmented liner installed immediately to prevent swelling. Stability problems close to the face may be encountered, especially in fault zones, and where there is strong water inflow. Rock instability in the crown can be mitigated by installing deformable filler material (e.g., pea gravel) behind the liner, while support can be increased by injecting grout into the filler (30% to 50%). With application of these methods since the 1970's, advance rates have improved from ~10 m/day to ~30 m/day, with larger excavated opening diameters.

A.3 Proposed Repository Layouts and Construction Methods

Oliver Heidbach (GFZ Potsdam) lectured on the state of *in situ* stress in northern Switzerland. The interpretation approach used is to model crustal deformability in 3D, then apply displacement (velocity) boundary conditions (e.g., -9 m/yr NW-SE). The stress state at any location consists of constant (e.g., gravitational), seismic (cyclic), and man-made components (excavation). Stick-slip friction on faults is an important factor. In general, the anticipated stress state in the Opalinus at repository depth will be nearly transversely isotropic with the maximum principal stress oriented vertically ($\sigma_1 = \sigma_V$), and the horizontal stresses equal ($\sigma_2 = \sigma_3 = \sigma_H$) and less than the vertical stress ($\sigma_H / \sigma_V \cong 0.8$).

Wulf Schubert (Technical University – Graz) gave a short presentation on repository design for rock and stress conditions. Openings should be perpendicular to foliation (including the strike of bedding) if possible to improve stability, limit displacements, and minimize the EDZ. Dipping strata are more difficult to design for than horizontal. The overall goal should be non-interference of repository drifts, through separation. If this is not achieved, stress redistribution in the rock mass may span multiple drifts, unloading some pillars and over-loading others.

Phillipe Nater (Pöyry Schweiz AG) presented an approach to repository underground design in the Opalinus and similar media. His introduction identified four excavations from which experience can be drawn: the Mont Terri URL, the historic Grenchenberg tunnel, excavations at the Konrad repository, and the Bure URL. The Grenchenberg single-track rail tunnel was completed in 1916 with a length of 8.6 km. It traverses the Opalinus claystone at approximately 700 m below the ground surface. The geology is strongly folded, and the Opalinus occurs five times with total exposure of 416 m. Rock conditions in the Opalinus were reported as good, and dry (unlike water inflow zones encountered elsewhere), but support requirements were relatively high. The lining consists of vaulted masonry up to 60 cm thick, and the invert was constructed as an arch with similar thickness. The tunnel has been abandoned. There are many tunnels through the Opalinus in Switzerland, and this one may be unique because of its age.

Repository drift sizes in claystone could range from 8 m^2 to 67 m^2 face area (for emplacement of a single HLW canister surrounded by backfill, up to a "vault" for intermediate-level and low-level waste (ILW/LLW). Spans from 3.2 m to 8 m can be expected. For openings excavated with a roadheader, the lining system would consist of immediate application of wire mesh and shotcrete, with yielding elements as needed (as for support of access tunnels in the Jura clay at Konrad). Immediate application is necessary to support construction operations, and to prevent damage from swelling or desiccation. Excavation and ground support should be designed to minimize the accumulation of voids in the EDZ. After deformation is allowed to occur, a final cover would consist of more shotcrete, applied over more wire mesh and steel sets where needed. Thicknesses would be increased wherever extra support is needed (e.g., greater burial depth, higher stress, larger spans, and/or longer service life). Rock bolts can be installed as needed.

Modeling of repository excavation performance is approached using six alternative constitutive models (representing different conditions: intact rock, fractured rock, interfaces, faults, large-scale structures, etc.). Three in-situ stress conditions are used (based on burial depth, varying the ratio of σ_v to σ_h parallel and perpendicular to the tunnel axis, with σ_v/σ_h ~1.5). Properties are estimated from available information (UCS, anisotropy, orientation to bedding, etc.). "Design levels" are used representing a range of conservatism from "most probable" through "ultimate" (unexpected conditions, beyond experience, or extended service life). Creep is not an issue for most applications because HLW disposal openings would be closed within ~5 yr, while ILW/LLW vaults would be closed in 20 to 30 yr. Only for service openings with operational lifetimes of 80 yr or longer, would creep be a design issue.

A final presentation was given by Gilles Armand (Andra) on rock mechanics of the COX argillite at the Bure URL, and consequences for the Cigéo repository design. The French repository is required to facilitate retrieval for at least 100 years, which means operational areas must remain stable for up to 150 years.

Emplacement openings for HLW will be long, horizontal borings excavated using a remotely operated mini-boring machine, and lined with continuous steel casing. Vault-type openings for ILW/LLW will be 9 to 11 m in diameter and up to 400 m long. Several support options have been identified for excavated openings, namely: 1) soft, with 3-m rock bolts and 8 cm of fiber-reinforced shotcrete; 2) medium, as above with additional 27 cm of cast-in-place non-reinforced concrete; and 3) maximum, with 3-m rock bolts and 45 cm of fiber-reinforced concrete applied in four layers.

Reported *in situ* stress conditions at the site are different from the Swiss Jura, with $\sigma_H/\sigma_v \sim 1.3$, and $\sigma_v \cong \sigma_h$. Observations in the URL show that for tunnels oriented parallel to σ_H, shear fractures are associated with inward block movement in the pillars. For openings parallel to σ_h, fractures in the roof and invert dip toward and away from the face, respectively. The favored orientation for stability of HLW borings is parallel to σ_H. All casings and liners need to be backfilled to accommodate anisotropic deformation.

Creep is defined as time dependent deformation with no change in pore pressure and no change in mean stress. Similar long-term deformation rates are observed at Mol, Bure, and Mont Terri suggesting a common mechanism. Data from Mont Terri *in situ* tests suggests this mechanism could be gradual pore pressure dissipation.