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Lake Mead Intake No 3 Tunnel - Geotechnical Aspects of TBM Operation

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Lake Mead Intake No 3 Tunnel - Geotechnical Aspects of TBM Operation

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ABSTRACT: The new Lake Mead No 3 Intake Tunnel will be constructed using a hybrid TBM (both slurry shield and open mode operation are possible) mostly through tertiary sedimentary rocks. Due to the very poor quality of the ground and the high pore pressures prevailing in the 4 km long subaqueous section of the tunnel (up to 14 bar, the highest pressures seen to date in closed shield tunneling worldwide), particular attention must be given to the risk of shield jamming or face collapse during boring or during the performance of maintenance activities in the working chamber. The paper outlines the expected geological-geotechnical conditions and discusses their potential impact on the operation of the hybrid TBM (e.g. mode of operation, face support pressures), as well as proposed auxiliary measures (e.g. advance drainage, grouting) and decision-making during construction.

PROJECT OVERVIEW

Lake Mead is located approximately 30 km east of Las Vegas behind the Hoover Dam (Figure 1). It supplies about 90% of Las Vegas valley's water. Over the last nine years, drought has caused the lake level to decline by more than 30 m. A further drop of the lake level may render the existing intakes unusable. In order to maintain the water supply, a third intake will be constructed about 40–60 m deeper than the existing two intakes, i.e. deep enough to function at the lowest lake levels (Feroz et al. 2007). The main structures of the new intake are a 170 m deep access shaft, an approximately 4,700 m long intake tunnel with an internal diameter of 6.10 m and an intake structure in the middle of the lake (Figure 1).

GEOLOGICAL CONDITIONS

The geology along the tunnel alignment has been explored by drilling 55 borings, 38 of them offshore. As shown in Figure 2a, the major part of the tunnel (including the subaqueous section) is located in tertiary sedimentary rocks of the so-called "Muddy Creek Formation" (conglomerates, breccias, sandstones, siltstones and gypsiferous mudstones of very variable quality). The tunnel alignment also crosses an older tertiary conglomerate of the Red Sandstone Unit, metamorphic rocks (amphibolites, schist and gneiss) and, close to the intake structure, basalts of the Callville Mesa Unit.

Furthermore, there are several faults in the project area. These are particularly critical, as they create the potential for water recharge directly from Lake Mead. Considerable water ingress must therefore be expected during construction. One well-known fault in the project area is the Detachment Fault, which has already been encountered

in the access shaft (Hurt et al. 2009). This fault is located at the beginning of the tunnel alignment and consists of strongly foliated Phyllonite with zones of crushed and brecciated rock. The tunnel will cross this fault over a length of about 50 m. The exploratory boreholes showed that the centre of the fault consists of a gravel-like cohesionless material (for about 10 m). Special attention has also to be given to the submerged continuation of the Las Vegas Wash, which will be crossed by the tunnel drive at a small depth beneath the lake bed.

The maximum depth beneath the current lake level is around 140 m. The rock cover decreases from its maximum of 170 m at the beginning to just 20–30 m in the last portion of the alignment (in the Las Vegas wash as well as close to the intake structure, see Figure 2a). Table 1 summarizes the most important parameters of the prevailing geological units.

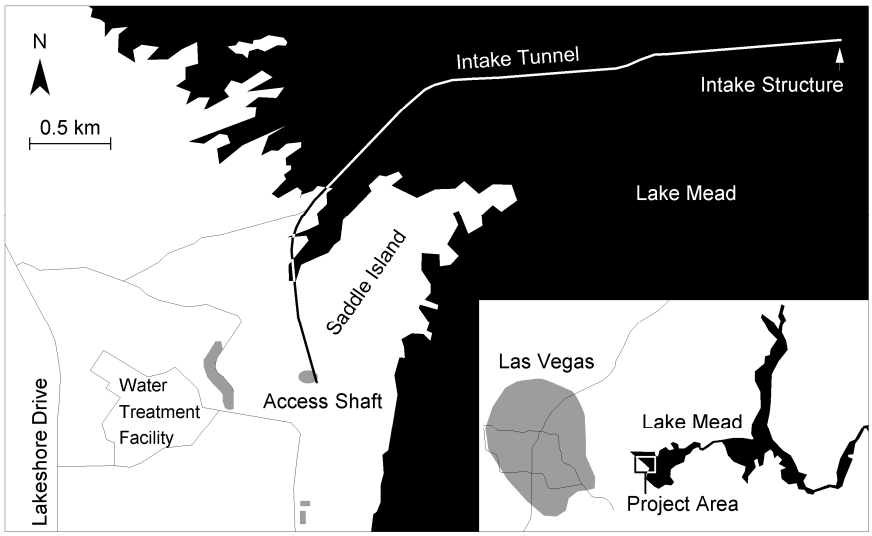

Figure 1. Project situation after Hurt et al. (2009).

Table 1. Ground parameters (from Vegas Tunnel Constructors, 2009).

Geological unit	Young's modulus E [GPa]	Cohesion c [kPa]	Friction angle φ [°]	Permeability k [m/s]
Saddle island lower plate (Pcl)	34 … 68	500 … 1500	35 … 40	$2\cdot10^{-9}$ … 10^{-6}
Saddle island detachment fault	7 … 14	0 … 40	25 … 30	$2\cdot10^{-6}$ … 10^{-5}
Saddle island upper plate (Pcu)	14 … 48	300 … 1000	35 … 40	$2\cdot10^{-8}$ … 10^{-6}
Muddy creek formation (Tmc 3)	1.4 … 2.8	50 … 300	26 … 35	10^{-10} … $3\cdot10^{-7}$
Muddy creek formation (Tmc 2)	0.3 … 1.4	50 … 300	26 … 35	$2\cdot10^{-9}$ … 10^{-7}
Muddy creek formation (Tmc 1 / Tmc 2)	0.7 … 2.8	50 … 300	26 … 35	10^{-9} … 10^{-7}
Tmc 1 to Tmc 2, fault zones	1.4 … 4.1	30 … 200	25 … 30	10^{-5}
Muddy creek formation (Tmc 4)	1.4 … 4.1	100 … 500	28 … 35	10^{-11} … 10^{-6}
Tmc 4 to Trs, fault zones	0.7 … 1.4	30 … 200	25 … 30	10^{-5}
Red sandstone unit (Trs)	1.4 … 3.4	30 … 150	25 … 28	$3\cdot10^{-9}$ … $2\cdot10^{-5}$
Pcu to Tmc 4, fault zone	1.4 … 4.1	30 … 200	25 … 30	10^{-8} … 10^{-6}
Tmc 4 beneath the Las Vegas Wash	1.4 … 4.1	30 … 200	25 … 30	10^{-8} … 10^{-6}
Calville mesa formation (basalts)	12 … 43	50 … 200	28 … 35	10^{-5} … 10^{-4}

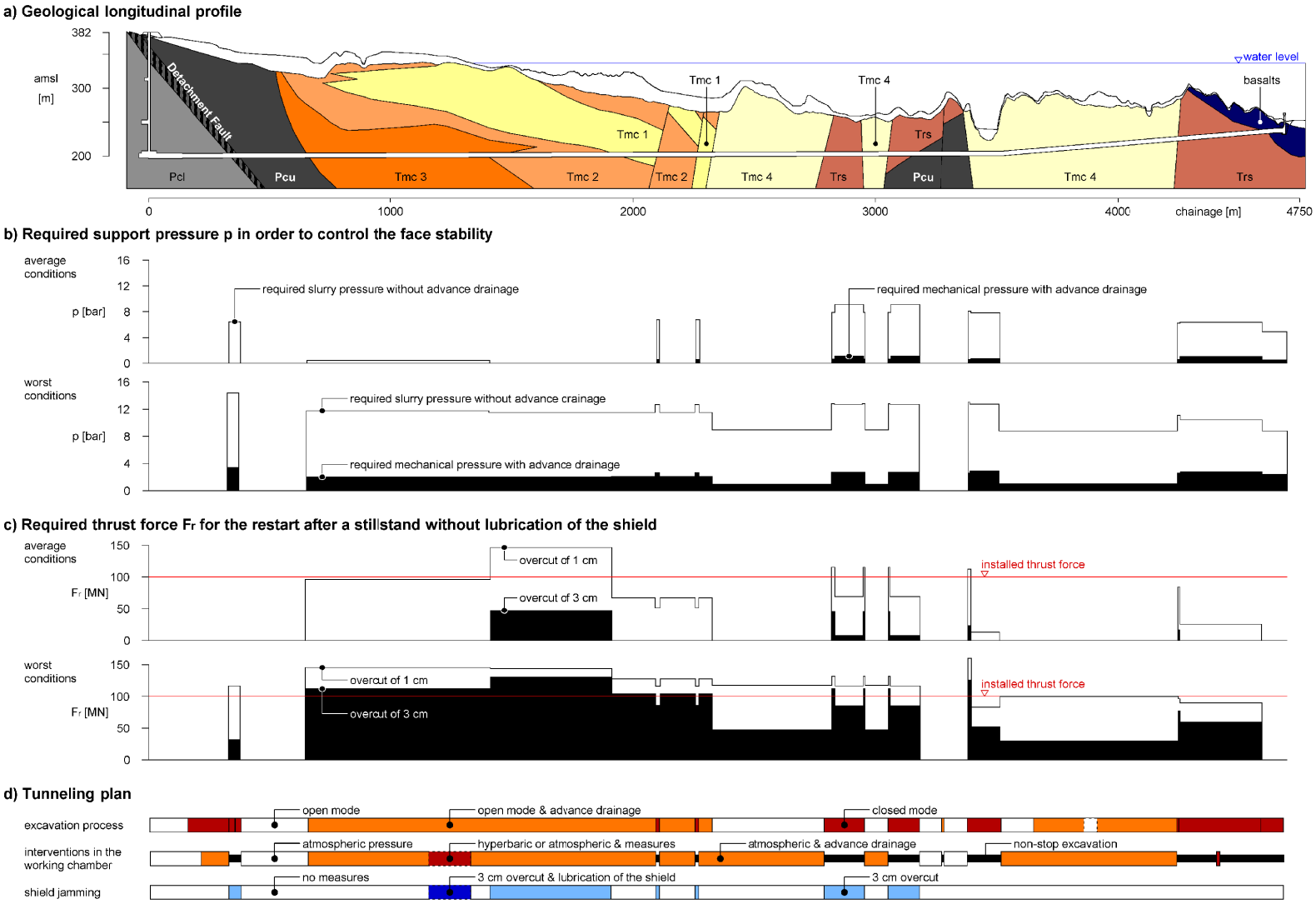

Figure 2. a) Geological longitudinal profile after Vegas Tunnel Constructors (2009); b) Required support pressure without (white columns) and with (black columns) drainage ahead of the face; c) Required thrust force in order to avoid shield jamming for overcuts of 1 cm (white columns) and 3 cm (black columns); d) Tunneling plan of the TBM including measures during the excavation process (top row), measures during interventions in the working chamber (middle row) and measures against shield jamming (bottom row). The dashed bars denote portions of a section where worse or better conditions may prevail than assumed for the rest of the section.

CONSTRUCTION METHOD

Due to the high hydrostatic pressures and the very variable quality of the sedimentary rocks prevailing over long portions of the alignment, attention was paid right from the start to the potential hazards of a cave-in at the working face or a flooding of the tunnel. The decision was therefore taken to construct the intake tunnel using a closed shield (Feroz et al. 2007).

The tunnel will be constructed using a convertible hybrid single shield TBM manufactured by Herrenknecht with a maximum installed thrust force of 100 MN (McDonald and Burger 2009). The TBM has a boring diameter of 7.22 m and can be operated either in open or in closed mode. In open mode, the face is not supported and a screw conveyor extracts the excavated rock from the working chamber (Figure 3a). In closed mode, the screw conveyor is retracted from the cutter head, mucking-out is done via the hydraulic circuit and the TBM supports the face with a pressurized bentonite slurry (Figure 3b). The TBM can be operated with partial, full or over-compensation of the water pressure and is designed to cope with hydrostatic pressures up to 17 bar – the highest ever pressures to date in closed shield tunneling worldwide. Due to the importance of advance probing and the possible need for pre-excavation ground improvement at least locally, the TBM is equipped with three permanent drill rigs and one mountable drill rig. Probing and drilling can also be carried out in closed mode using a blow-out preventer unit (McDonald and Burger 2009). Nevertheless, even if the TBM allows for boring in closed mode, the high hydrostatic pressures will make it extremely difficult to perform inspections and maintenance activities in the working chamber. In order to ensure stability during interventions, the face will have to be supported by applying compressed air. At the high pressures that are expected, however, professional divers will be required to perform the hyperbaric interventions and this will be very time-consuming. In addition, the stretches with closed-mode operation must be kept short because closed-mode operation generally results in lower TBM performances.

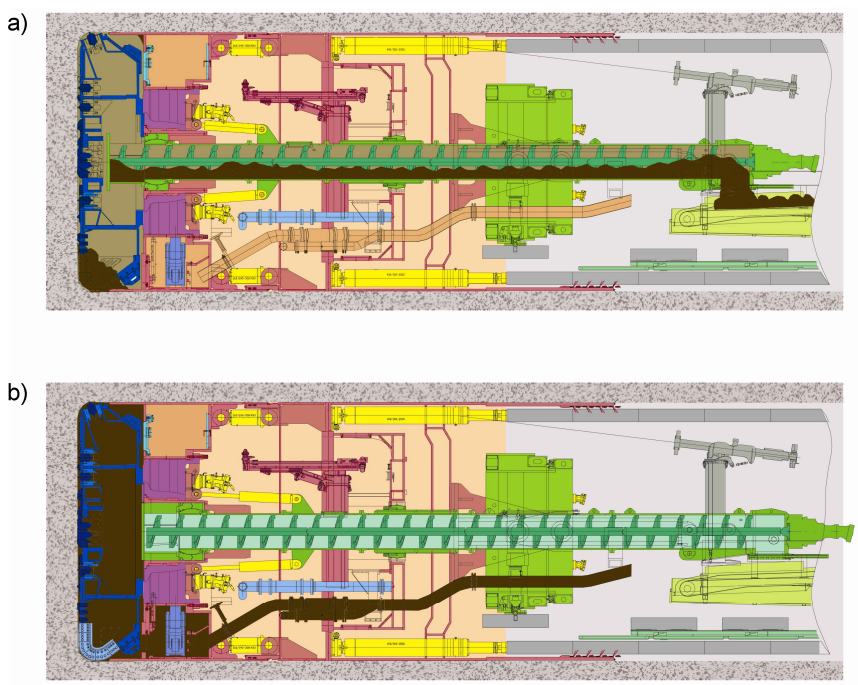

Figure 3. TBM configuration for open (a) and closed (b) mode operation after McDonald and Burger (2009).

These considerations, in combination with the lack of experience with closed-mode TBM operation at such high hydrostatic pressures, made it necessary to conduct an investigation into the limits of open mode operation, i.e. working under atmospheric pressure in the chamber, possibly in combination with auxiliary measures such as grouting or drainage.

POTENTIAL HAZARDS

A high damage potential, relatively high pore pressures and limited accessibility in the pre-construction phase are main features of subaqueous tunnels (Anagnostou 2009).

The high damage potential results from the possibility of a complete flooding of the tunnel in the case of a hydraulic connection to the lake. The risks associated with large water inflows can be mitigated to a large degree by installing extensive pumping capabilities and through a TBM design that will allow rapid conversion from open mode to closed mode by installing a screw conveyor for the removal of the excavated ground in open mode.

The high hydrostatic head leads, in combination with the small depth of cover in places, to the development of high seepage forces that increase the risk of face instability in a low strength ground. A collapse of the working face represents the most serious hazard scenario in the present case. Furthermore, given the sedimentary character of the prevailing rocks, jamming of the shield due to squeezing (Ramoni and Anagnostou 2009a) represents an additional hazard scenario.

The next two sections of this paper concern the geomechanical calculations and the assessment of these two potential hazards. For the purpose of assessing tunneling conditions, the tunnel has been subdivided in sections with practically uniform conditions. In order to check the sensitivity of the results, all of the calculations were performed for three sets of parameters, representing the so called "best", "average" and "worst" conditions for each section.

COLLAPSE OF THE WORKING FACE

Details of the face stability assessment for the Lake Mead Intake No 3 Tunnel can be found in Anagnostou et al. (2010). Here only the most important assumptions, geotechnical considerations and investigation results will be presented.

With the exception of tunneling through cohesionless, granular soil, the stability of the tunnel face is in general time-dependent, i.e. a face that is stable in the short-term may collapse in the long-term. The time-dependency can be traced back to the rheological behavior of the ground (tertiary creep) or to the generation and subsequent dissipation of excess pore pressures (consolidation, cf., e.g., Anagnostou 2007b). The latter is particularly relevant in the case of low-permeability sedimentary rocks. The short-term (so-called "undrained") conditions are more favorable than the so-called "drained" conditions which affect the long-term behavior of the ground and are characterized by the development of destabilizing seepage forces.

In general, the less permeable the ground, the more rapid the excavation and the shorter the standstills, the more reasonable it is to assume favorable short-term conditions. In the case of high ground permeability, no favorable short-term behavior can be observed and unfavorable drained conditions will prevail in the face area already during excavation (Ramoni and Anagnostou 2007). The influence of ground permeability k on the distinction between "undrained" and "drained" conditions during TBM excavation has been studied with numerical calculations that simultaneously take in account both the stress re-distribution and the consolidation process around the advancing

tunnel heading (Anagnostou 2007a). Assuming an average TBM advance rate of 10 m/day, the computational results indicate that favorable undrained conditions apply only where there is low ground permeability ($k \leq 10^{-8}$ m/s) and only during the excavation process, including short standstills of up to 0.5–1 day (Anagnostou et al. 2009). For higher permeabilities or for longer standstills, unfavorable drained conditions must be expected. Over long portions of the alignment, the expected range of ground permeabilities (Table 1) is in the geotechnically demanding transition zone between drained and the undrained conditions. Face stability analyses have been carried out for both conditions, and the prediction uncertainties that exist with regard to the time-dependency of the ground behavior were taken into account in the tunneling plan.

Short-term face stability was investigated with the computational model of Anagnostou and Kovári (1994), while the calculations concerning long-term face stability were made by applying the nomograms of Anagnostou and Kovári (1996). In both cases, the assumed three-dimensional collapse mechanism consists of a wedge ahead of the tunnel face and an overlying prism (both in a state of limit equilibrium). In short-term the stability of the face is governed by the undrained shear strength of the ground, in long-term by the effective strength parameters (c' and φ'). The calculations showed that for the given range of ground parameters the face would be stable in the short-term over the entire tunnel alignment. The white columns in Figure 2b apply to long-term stability conditions and show the minimum slurry pressure (or compressed air pressure) required in the working chamber in order to avoid face instability in the absence of a mechanical support. In long-term (which, as mentioned above, concern, e.g., a standstill longer than 1 day or a ground permeability higher than about 10^{-8} m/s), for both the "average" and "worst" ground strength parameters, closed mode operation with a stabilizing slurry pressure would be necessary for an extended portion of the alignment.

Operation and maintenance in closed mode at high pressures are very demanding and result in low advance rates. In order to operate the TBM in open mode over long portions of the tunnel, additional measures are necessary. With this in mind, we investigated whether advance drainage of the ground ahead of the tunnel face would result in significantly greater stability of an unsupported face.

Advance drainage – which, in the present case, can be carried out by means of boreholes drilled in the tunnel face through the cutter head – reduces pore pressures and their gradients in the core ahead of the face and thus also reduces the destabilizing seepage forces acting within the ground towards the opening. Once again, the computations were based upon the limit equilibrium mechanism proposed in Anagnostou and Kovári (1994). Seepage flow was taken into account by introducing the seepage forces into the equilibrium equations. In order to estimate the seepage forces, three-dimensional, transient seepage flow calculations were carried out with the Finite Element Code COMSOL Multiphysics (formerly FEMLAB; COMSOL 2009), taking into account the incomplete drainage of the ground due to time-effects (drainage takes more or less time depending on the permeability of the ground) and due to the spacing of the drainage boreholes. Figure 4a illustrates the effect of drainage of the core on the hydraulic head field in a cross-section 2 m ahead of the tunnel face (Section A-A in Figure 4b). The figures on the left apply to the case of "natural" drainage through the open face, while the figures on the right apply to the case of drainage via six horizontal boreholes drilled in the upper part of the tunnel face. The drill pattern was selected according to McDonald and Burger (2009). As shown in Figure 4a (where darker tones apply for a lower hydraulic head H), the pore water pressures within the ground ahead of the face can be reduced significantly by advance drainage. Such a reduction also leads to lower seepage forces acting on the potentially unstable wedge in front of the atmospherical tunnel face. Figure 4b shows the average water pressure acting upon a potentially unstable wedge. For the present case, drainage over four hours by six boreholes halves the pore pressure acting on the wedge.

The reduction in pore pressures observed in the ground ahead of the face is very helpful in terms of face stability. The black columns in Figure 2b show the necessary mechanical support pressure in the case of advance drainage by six boreholes (under atmospheric conditions in the working chamber). The needed support pressure is significantly lower than the slurry pressure which would be needed in the absence of advance drainage (white columns). According to Figure 2b, advance drainage represents a very powerful improvement method and extends the feasibility range of open mode operation.

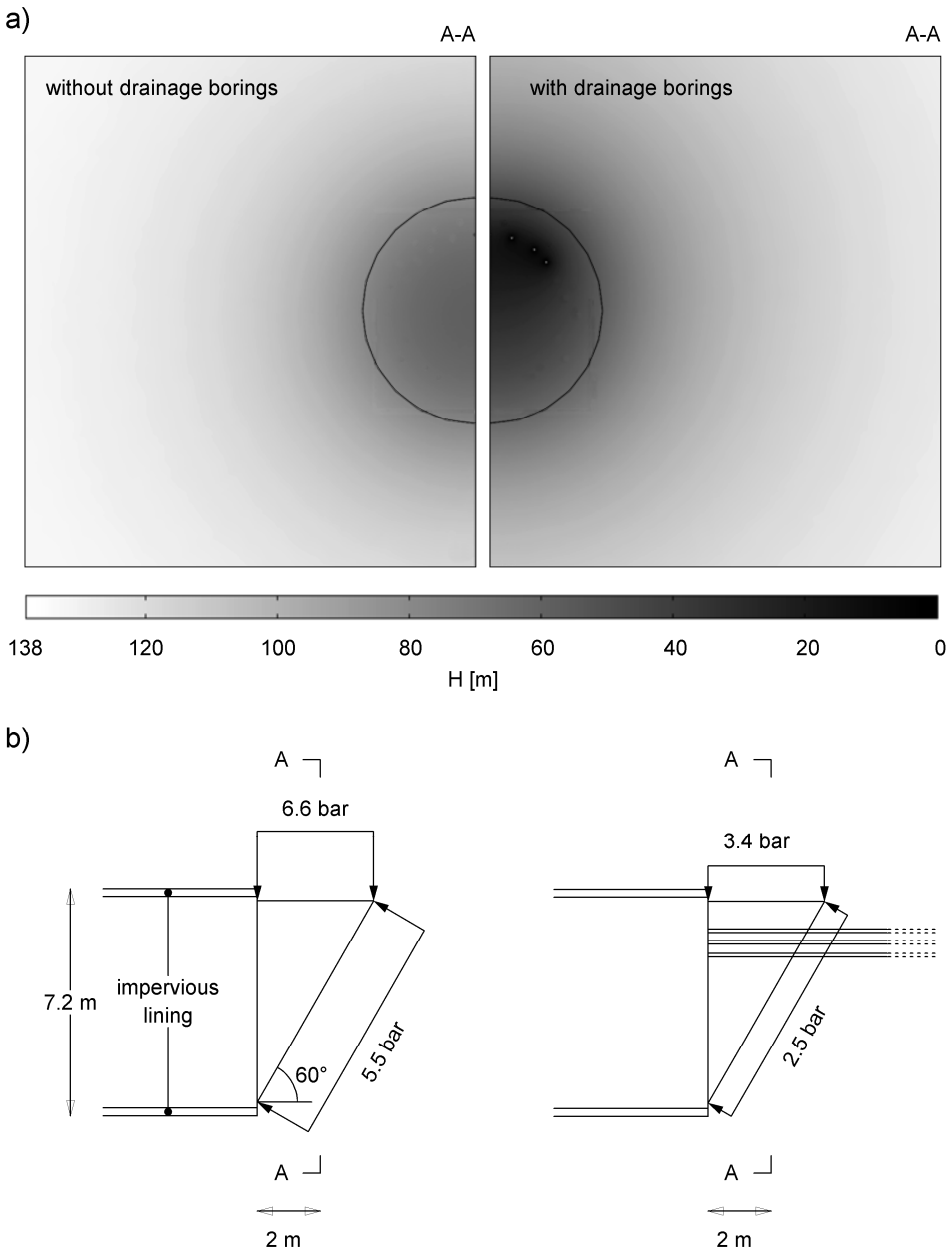

Figure 4. a) Distribution of the hydraulic head *H* at the cross section A-A for the cases of "natural drainage" (left) and drainage using six boreholes ahead of the face (right). Both cases apply for a drainage time of *t* = 4 h assuming a permeability of the ground of $k = 10^{-7}$ m/s and a storage coefficient of $s = 1.3*10^{-5}$ m^{-1}; b) Average pore pressure acting on a potentially unstable wedge (sliding plane inclined by 60°) for the two cases.

JAMMING OF THE SHIELD DUE TO SQUEEZING

When using a TBM, relatively small convergences (in the order of one or two decimeters) may lead to considerable difficulties, due to the geometrical constraints of the equipment. On account of the poor ground conditions that are expected along some stretches of the alignment, jamming of the shield due to squeezing ground could therefore not be excluded a priori and it was accordingly investigated computationally.

The hazard scenario was assessed by computing the thrust force required for each tunnel section under uniform conditions. The calculations were carried out systematically for different operational modes, stages and measures. More specifically they were performed: (i) both for open and closed mode operations (the latter assuming full compensation of water pressure); (ii) both for restart after a standstill (static skin friction) and for ongoing excavation (lower sliding skin friction, but additional cutter head force for boring taken into account); (iii) with and without lubrication of the shield extrados (lubrication reduces the friction by about 50% and is automatically applied in the case of closed mode operations with bentonite suspension supporting the face); (iv) three values (3, 2 and 1 cm) for the radial gap size between shield and ground in order to study the effects of a reduction in the overcut (caused by the wear of the gauge cutters or by the packing of fines between the shield and the ground). The positive effects of a possible delayed ground response (i.e., time-dependent behavior due to consolidation or creep) were not taken into account in the calculations. This is a reasonable simplification in view of the difficulty of making a reliable forecast of the time-dependent development of ground deformations.

Concerning open mode operation, the required thrust force F_r was taken as equal to:

$$F_r = F_b + F_f , \qquad (1)$$

where F_b is the boring thrust force and F_f the thrust force required for overcoming shield skin friction considering the friction coefficient $\mu = 0.30$ for sliding friction and $\mu = 0.45$ for static friction, respectively (Gehring 1996). The boring thrust force F_b was considered only for the operational stage of "ongoing excavation" and assumed as:

$$F_b = F_c \, n_c = 13 \, MN , \qquad (2)$$

where $F_c = 267$ kN is the bearing capacity of the cutters after Wehrmeyer et al. (2001) and $n_c = 48$ is the number of cutters.

The calculation of the thrust force required during closed mode operation has to consider additionally the thrust force required due to the face support pressure. Therefore, Equation 1 has to be enhanced with the additional term F_p:

$$F_r = F_b + F_f + F_p , \qquad (3)$$

where F_p is equal to the integration of the support pressure over the face. Assuming full compensation of the water pressure,

$$F_p = H_w \, \gamma_w \, \pi D^2/4 , \qquad (4)$$

where H_w is the depth of the tunnel beneath the lake level or groundwater table, γ_w the unit weight of the water and D the boring diameter.

Another difference between open and closed mode operation concerns initial stress, which has been considered in the calculations. For open mode operation, total stress shall be considered (cf. Anagnostou and Kovári 2003), i.e.

$\sigma_0 = H\,\gamma' + H_w\,\gamma_w \quad if\ H_w > H$ (subaqueous portion), and

$$\sigma_0 = H_w\,\gamma' + (H - H_w)\,\gamma_d + H_w\,\gamma_w \quad if\ H_w < H \text{ (land portion)}, \tag{5}$$

where H, γ' and γ_d denote the depth of cover, the submerged unit weight and the dry unit weight of the ground, respectively. For closed mode operation, the effective rather than the total initial stress must be taken into account:

$\sigma_0 = H\,\gamma' \quad if\ H_w > H$ (subaqueous portion), and

$$\sigma_0 = H_w\,\gamma' + (H - H_w)\,\gamma_d \quad if\ H_w < H \text{ (land portion)}. \tag{6}$$

The effective initial stress is lower than the total initial stress (which is favorable and leads to a lower frictional resistance F_f) but on the other hand the face support pressure must also be taken into account (F_p in Equation 3).

A total of 1512 input parameter sets has been considered in the calculations for the required thrust force (Anagnostou et al. 2009). It was possible to make such a comprehensive investigation only on the basis of the design nomograms presented by Ramoni and Anagnostou (2009b). These nomograms assume a constant overcut along the shield, while the actual shield becomes smaller stepwise. This simplifying assumption tends to be unsafe concerning the loading of the front portion of the shield but is generally safe for the rear shield, which is the most critical part of the machine with respect to jamming.

Figure 5b shows the convergence Δu of the bored profile according to a comparative numerical calculation with a more realistic modeling of the actual shield geometry (Figure 5c). Due to the conicity of the shield, the gap between ground and shield (dashed lines in Figure 5b) becomes closed three times: in the front part of the shield at a distance of about 2 m from the working face and, later, also in the middle and in the rear part of the shield. When the ground establishes contact with the shield by closing the gap, a pressure p develops upon the shield. The thrust force required to overcome shield skin friction can be calculated by integrating the ground pressure over the shield surface and taking into account the skin friction coefficient. The simplified computational model of the nomograms is generally safe concerning the required thrust force.

Figure 2c shows the required thrust force over the entire tunnel alignment for the case of restarting after a standstill during open mode operation without lubrication of the shield. The results are presented both for the average and for the worst parameter combinations. The effect of the amount of overcut is illustrated by the black and the white columns (for 3 cm and 1 cm radial gap sizes, respectively). The positive effects of lubricating the shield can easily be understood if we bear in mind that the thrust force required to overcome shield skin friction depends linearly on the assumed skin friction coefficient μ between shield and ground.

The results of the computational investigations described in this section indicate that the potential problem of shield jamming is far less critical than that of face instability but must be taken into account in isolated portions of the alignment. As discussed later, the application of standard counter-measures is anticipated, such as lubrication of the shield mantle and the installation of new gauge cutters (in order to assure enough clearance between shield and ground) before entering the critical stretches.

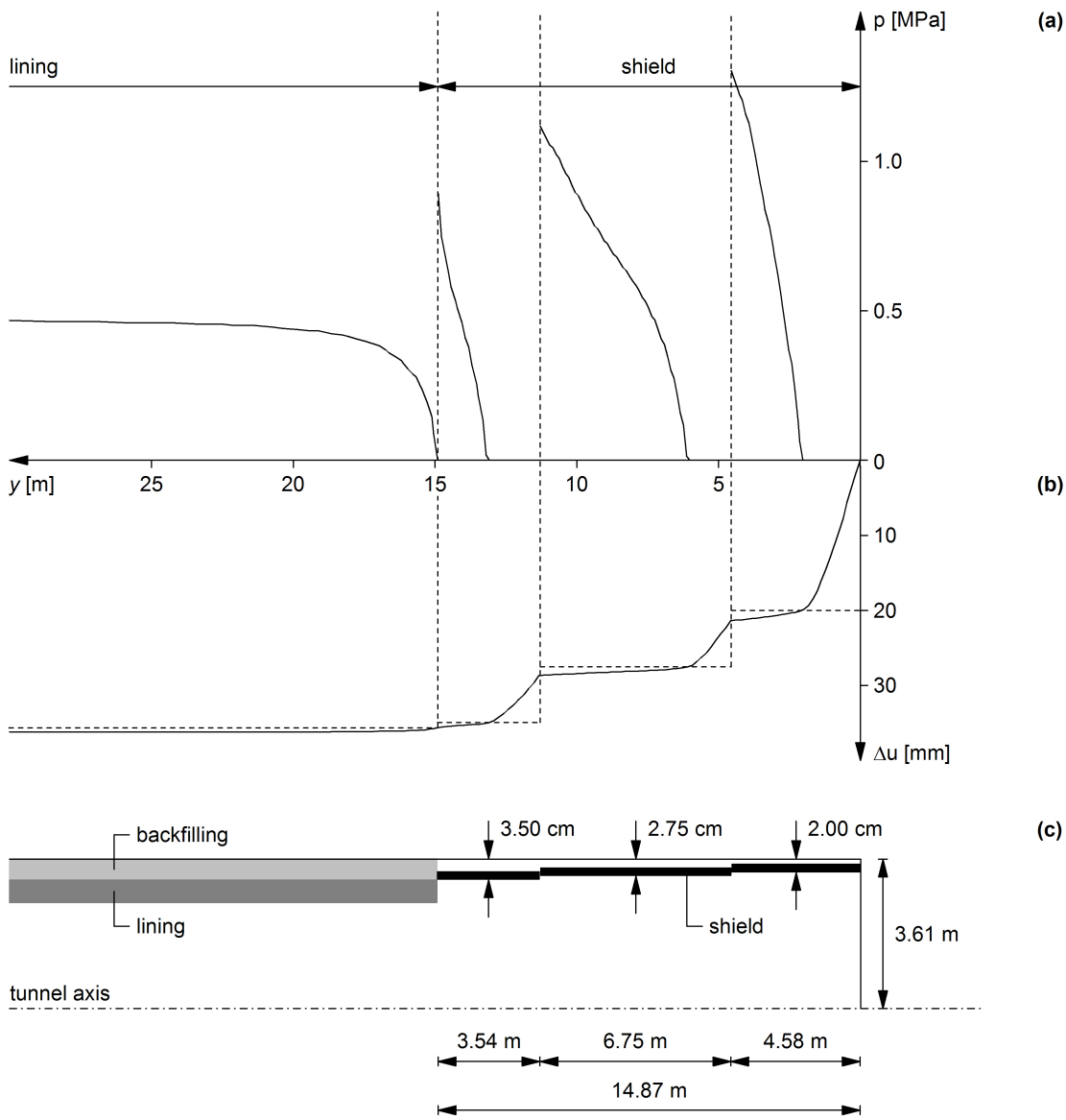

Figure 5. Ground pressure *p* acting upon the shield and the lining (a) and convergences Δ*u* (b) for the actual geometry of the shield (c); the computations apply for the Conglomerate of the Muddy Creek Formation ("worst" conditions).

TUNNELING PLAN

The tunneling plan defines the TBM operational modes (open or closed), the operating pressures and whatever auxiliary measures are required. It is based upon a qualitative evaluation of the geological profile, the geomechanical calculations mentioned above, engineering judgment and risk considerations. As is the case for any tunneling project, there are uncertainties with respect to, (i), the structure of the formations (e.g. the sequence of the lithological units and the extent and location of fault zones) and, (ii), the response of the ground to tunneling operations (e.g. the stand-up time of the ground or the intensity of excavation-induced convergences).

The consequences of type (i) uncertainties can be reduced by systematic advance probing during the TBM drive. On the whole, advance probing is recommended for the entire tunnel. At each drilling station, two boreholes without

core recovery shall be drilled. The timely and reliable identification of critical zones will also necessitate, however, core drilling on some occasions. A reliable geological pre-exploration of the conditions prevailing ahead of the face will reduce the need for precautions (such as closed mode operation). If the advance probing is less reliable, a greater number of protective measures will be required in order to handle risks which will possibly never materialize. The data from percussive drillings (water quantities and drilling data such as penetration rate, penetration force and torque during the drilling process) is relevant for determining water circulation or the presence of sharp transitions between hard rock and soft ground and it is therefore reliable only with respect to specific geological features (for example, highly fractured water bearing zones).

Even in the case of a well-known sequence of geological formations, there can be uncertainties with respect to ground behavior, i.e. the above-mentioned type (ii) uncertainties. In the present project, such uncertainties are relatively large due to the character of the ground and, more specifically, the difficulties of assessing the effects of the "time" factor. As already mentioned, for the expected range of ground permeabilities long portions of the alignment fall into the geotechnically-demanding intermediate stage between so-called "drained conditions" and so-called "undrained conditions". In this intermediate stage it cannot be said with certainty whether favorable short-term conditions or unfavorable long-term conditions will apply. This introduces an element of uncertainty concerning the stand-up time of the tunnel face and has therefore a direct consequence for the operating mode of the machine. The geomechanical calculations indicate that the effect of this uncertainty can be reduced significantly (but not entirely) by advance drainage.

According to Figure 2b, assuming that the "average" conditions prevail over the entire alignment, a pressurized face would be necessary over about 35% of the tunnel length. In the case of the "worst" conditions (a highly improbable hypothesis, of course) this figure increases to about 85%. The advance drainage of the ground reduces the amount of support required considerably. For the "average" conditions, only the faults, the Red Sandstones and the basalts would require an additional mechanical face support of only 0.5–1.1 bars. Assuming "worst" conditions a mechanical face support of 0.9–3.4 bars is required (instead of the 9–14 bars of slurry pressure). This emphasizes the huge importance of careful ground evaluation and decision-making during construction.

The top row of Figure 2d gives an overview of TBM operational modes for the excavation process which can reasonably be assumed taking into account the information available at present. In order to mitigate the risk of a face collapse, the faults, the Red Sandstones and the basalts have to be excavated in closed mode (red bars). Considerable slurry loss and a subsequent loss of the support pressure in the gravel like core of the Detachment Fault must be avoided by operating the TBM in closed mode in combination with advance ground improvement by grouting. The remaining portion of the tunnel alignment can either be excavated in open mode (white bars) or in a combination of open mode with advance drainage of the ground ahead of the face (orange bars). Figure 2b indicates that the face would not be stable in the long-term for the "worst-case" strength parameters, but one should consider that the adverse combination of high permeability ($k > 10^{-8}$ m/s) and low strength is rather improbable as low strength values apply to the more clayey units which exhibit rather low permeabilities. One may consider, furthermore, that the risk of face instability during ongoing excavation may be acceptable (no people in the working chamber) as long there is no connection to the lake (sufficient depth of cover, low-permeability ground).

Nevertheless, during the performance of inspections and maintenance activities in the working chamber the risk of a face collapse is clearly unacceptable. Due to the uncertainties concerning high water pressures, hyperbaric interventions should be avoided and continuous (non-stop) TBM operation in closed mode is recommended in the

most critical stretches (black bars in the middle row of Figure 2d). Continuous operation should be possible at least for the shorter critical portions, provided that careful maintenance is carried out just before entering these stretches. In the relatively long portion through the Red Sandstone Unit and the basalts at the end of the tunnel alignment, however, one or more maintenance stops will probably be necessary. The work will then have to be carried out either under hyperbaric pressure or after grouting the ground (red bar). The possibility of non-stop excavation in this particularly adverse tunnel portion will be re-evaluated later, taking account of the experience gained during the TBM drive. In the remaining portions of the tunnel alignment, the interventions in the working chamber can be performed in open mode. In the metamorphic rock sections the working chamber can be accessed without any additional measures (white bars), whereas in the tertiary sedimentary rocks the working chamber can be accessed only after finding an appropriate location and after draining the ground ahead of the face (orange bars). For these stretches, it is recommended first of all that the face be inspected and its stability evaluated. If the ground conditions are good enough, drainage shall be carried out before entering the chamber. In the case of adverse face conditions, the TBM drive should be continued and then stopped again after few meters for a new inspection and assessment of the face. Based upon the frequency and extent of poor rock intervals found in the exploratory boreholes, it is reasonable to expect that one will find an appropriate location after one or two restarts. However, the possibility of longer stretches with poor ground conditions cannot be excluded entirely. If a safe location cannot be found after a number of stops and restarts, core drilling is recommended in order to find a safe spot for maintenance work. Where it is not possible to identify such a place with sufficient reliability, measures such as face bolts or grouting will be necessary in addition to drainage, in order to ensure the stability of the face (the alternative is for divers to perform the work under hyperbaric conditions).

Regarding the potential hazard of shield jamming due to squeezing, the major portion of the tunnel alignment can be excavated without taking any measures (see white bars in the lower row of Figure 2d). An overcut of at least 3 cm has to be provided in the so-called "Tmc 2" unit of the Muddy Creek Formation, in the Red Sandstones Unit and in most of the fault zones (light blue bars). Regarding the overcut of 3 cm, it should be noted that, due to the packing of fines or gauge cutter wear, the actual gap may be lower than the theoretical one. A lubrication of the shield in open mode will be necessary only if the TBM has to be stopped and restarted in the very low quality ground of the Muddy Creek Formation (dark blue bars). As already mentioned, this can be avoided through the use of core-drilling to identify appropriate locations for maintenance work.

FINAL REMARKS

According to our investigations, a considerable portion of the tunnel can be constructed by open-mode TBM operation in combination with advance drainage of the ground ahead of the face and systematic advance probing, including core-drilling on some occasions. The risk of shield-jamming is less critical than face instability but must be taken into account, particularly in the sedimentary rocks before the Las Vegas Wash.

The recommended operational modes are reasonable from the perspective of a qualitative risk analysis (the risk of an undesired event occurring is considered acceptable if its impact or probability of occurrence are small). A systematic evaluation of the experience gained during the TBM drive will enable better management of the uncertainties concerning ground behavior.

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