A technical note on stabilization and closure design of a salt mine in Detroit, USA

NASIM UDDIN
Department of Civil and Environmental Engineering, University of Alabama at Birmingham, AL, 352984

Abstract

Lack of proper maintenance and repair of Detroit Salt Mine, during the life of the 90-year old facility and an aggressive subsurface environment, presented many unusual design issues. The technical note describes how the mine stabilization and closure design addressed the issues and ensured that ground water was controlled at all phases of the construction. It introduces the design challenges for the stabilization and the closure of the Salt Mine which includes design of the stabilization measures in the mine to ensure long term stability, and concrete plugs in two mine shafts to prevent long-term leakage into the mine and subsequent solution of the salt. The design includes grouting in and around the shaft plugs including two-state cement and chemical curtain grouting in the rock surrounding the plug and high-pressure contact grouting at the concrete-soil and concrete-rock interface. The paper addresses the selection of plug locations, plug geometry, concrete mix design, cooling of the concrete and grouting of the rock and rock/concrete interface.

Introduction

Salt production at the Detroit Mine stopped in 1983. Since then various attempts have been made to find a use for the mine, including storage of waste products, natural gas and compressed air for power generation. However, these uses for the mine have not proved economically feasible, and in order to minimize future maintenance of the mine, the owner hired Acres International to design a closure system for the mine. The room and pillar mine workings are stable with no signs of salt movements or rock distress. However, significant leakage occurs at the shafts. The main objectives for the closure design was to seal the shafts and prevent water inflow into the mine. Long term seepage into the closed mine could result in solution of the salt and progressive collapse, potentially to the surface.

The intent of the design of the closure system was to seal the shafts to prevent leakage into the mine and preserve the current mine stability. Closure of the mine will eliminate the long-term maintenance and repairs needed to provide safe access for inspection and pumping of water from the mine.

Mine development

The Detroit Mine is located in the Detroit, Allen Part and Melvindale Townships south of the River Rouge in Detroit. The mine is adjacent to a large Ford Motor Company Plant and extends under a mixture of residential and heavy industrial properties. A major railroad crosses above the mine in a northeast direction. The mine construction was started in 1906 with the sinking of Shaft No. 1 which took five years to complete. Very difficult ground and artesian water conditions were encountered as described in a paper by Fay (1911).

Construction of a second shaft followed, believed to have been completed in 1922. Over the next 8 to 10 years, extensive repairs were performed on both shafts. Two 42-inch diameter (1.07m) pipes were installed in shaft No. 1 and the shaft was backfilled around the pipes with concrete.

It is believed that this was an attempt to control leakage into the shaft. Extensive repairs were made to the No. 2 shaft. Sections of the concrete lining were replaced and at some time later, a brick lining was placed in the shaft. The No. 1 shaft was used for backup and emergency personnel access with a two-level man cage in each of the 42-inch (1.07m) diameter inner shaft pipes. No. 2 shaft was used for hauling salt. The bottom of the No. 2 shaft was equipped with loading pockets. After some initial mining at the A salt level, salt production was switched to the B level and continued at that level for the remainder of the mine operation. Salt was mined from about 1910 to 1983 by a room and pillar method with about 50 percent extraction. The layout of the mined areas is irregular, to suit mining rights and sensitive property, such as a railroad corridor. The mine covers approximately two square miles. The mine is dry with no water leakage except at the shafts. Some pools of water occur within the mine. The water is thought to be associated with condensation from the ventilation system or connate water from the salt itself.

Regional and site geology

The Detroit Salt Mine is located near the southeastern edge of the Michigan Basin. The Michigan Basin is a broad, subsidence structure that covers most of the state of Michigan. It is bounded by platformal sedimentary rock formations to the west, south and east. To the north it is bounded by the Canadian Shield, consisting of metamorphic and igneous rocks. The platformal rock formations are divided into the Ohio Basin and the Appalachian Basin that lie to the east and southeast of the Michigan Basin. The basin is distinct areas divided by arches where relatively little or no subsidence occurred. The Findlay Arch that lies to the south of the Detroit Salt Mine divides the Michigan Basin from the Appalachian Basin.

The development of these basins contains many common...
rock formations, built of particular relevance to this project is the presence of the Salina group, which is a thick series of bedded salts, carbonates and evaporates.

The geology of the immediate area of the Detroit Salt Mine was derived from logs of the mineshafts, exploratory core holes and nearby oil, gas and brine wells. Figs. 1 and 2 show the generalized geological section of the strata underlying and penetrated by the excavation of the No. 1 and No. 2 shafts. At the mine site, surface geology consists of 86 ft (26.2 m) of unconsolidated glacial deposits (mostly blue clay with scattered cobbles and boulders). Underlying the glacial deposits is approximately 330 ft (100.6 m) of Devonian limestone and dolomite. Next below is 113 ft (34.5 m) of water bearing Sylvania sandstone; below which lies another 344 ft (105 m) of dolomite that includes a few shaley beds toward the base. Next below is approximately 1,000 ft (305 m) thickness of inter bedded dolomite, salt, shale and anhydrites characteristics of the Salina Group evaporates. The salt units at the mine generally strike to the ENE and dip or slope gently to the north at approximately 1.4 ft (0.43 m) per 100 ft (30.5 m).

**Groundwater**

The groundwater hydrology in the vicinity of the Detroit Salt Mine was determined from logs of the mineshafts and most recently in the drilling of DH 01, a salt disposal well installed in 1994. From the surface down to a depth of 680 ft (207.4 m), several major water-bearing zones were encountered. Artesian conditions are encountered at depths of 85 ft (26 m) to 100 ft (30.5 m) (top of rock) and 170 (52 m) to 190 ft (58 m) (Lucas Dolomites). The groundwater is generally of poor quality and highly charged H$_2$S at depths below 980 ft (24.4 m). Seepage of groundwater into the shaft from the shaft penetrations is currently collected by a system of gravity feed lines into sumps and tanks at mine level and subsequently pumped to the ground surface for disposal.

Present leakage rates are 30 gpm (113 liters/minute) for shaft No. 1 and 20 gpm (71 liters/minute) for shaft No. 2. Periodic measurements indicate that these rates are increasing. Historical data indicates that the aquifers penetrated by the shafts are under artesian head of about 20 ft (6.1 m) as measured at the ground surface. This is consistent with recent report (Raven 1992) of "super
normal” pressure up to 1.7 times hydrostatic pressure measured in similar rock strata in Southern Ontario. Artesian flows were encountered in borehole DH-1 where flows to the surface of up to 1000 gpm (3790 liters/minute) were estimated at a hole depth of 170 ft (52 m).

Mine closure design
The primary objective in the design of the mine closure was to seal the shafts and prevent leakage into the mine, which would cause solution of the salt and result in mine instability. To meet this primary objective, it was decided to construct concrete plugs in each shaft. The plugs were located as high as possible in the shaft to minimize the hydrostatic head on the plugs at the same time ensuring that the plugs were below any potential leakage zones. The plug location selected was within a competent dolomite and shale stratum 300 ft (91.5 m) above mine level.

Because of the possibility of artesian flow from the shaft once the lower plugs are installed, a second plug is required at the top of the shaft. The upper plugs also serve as protection caps to the top of the shafts.

Factors considered in the design of plugs
The following factors are considered in the design of the underground plugs (Auld 1982, Flugge 1967, Jaeger et al 1979, Lancaster 1964, Manning 1961, Timoshenko et al 1961): The purpose for which the plug is to be constructed, the type of excavation in which the plugs are to be installed, where the plugs are to be sited in relation to the prevailing rock and working conditions, plug shape, hydrostatic head on the plug, the condition of and the stress in the rock surrounding the plug, the strength of, and stresses in, the material of the plug; and the method of plug construction. One of the most important factors considered in deciding
where to place a plug is the condition of the surrounding rock. Detailed geologic evaluations are carried out to locate the plug in the rock, which is free from geological disturbances which could provide a leakage path for water, not in or near the fracture zones of highly stressed ground resulting from mining excavations; and not in the ground likely to be affected by subsequent ground movement resulting from external disturbances. Figs. 1 and 2 show the arrangement of the upper and lower plugs.

This concrete plug is un-reinforced and incorporates a taper to ensure compressive stress over the entire rock/concrete area. The resistance to the applied hydrostatic pressure is achieved through mechanical interlock with the rough excavation face of the surrounding rock. Garret and Campbell Pitt (1961) considered plug length to be governed more by leakage resistance around the sides and through the surrounding rock than by structural strength. The long plug length required for leakage resistance also ensures low shear and bearing stresses at the concrete-rock interface. The plug design required that the concrete lining be completely removed to ensure that leakage paths at the rock-lining interface were intercepted. The rock excavation for the tapered plug also ensured that rock disturbed by the blasting for the original shaft excavation would be removed.

Factors considered when evaluating the stresses in, and strength of, the plugs include concrete compressive strength, the early state development of strength, the shear or bearing stress at the plugs to rock interface, the pore water pressure in the concrete, and the possible end spalling of the plug due to high stresses set up by ground pressure. Early state development strength is very important because it is essential that plugs develop their specified strength without any detrimental effects occurring from shrinkage, thermal changes or ground pressure. Steps are, therefore, taken to overcome these factors, to protect the integrity of the concrete and to minimize leakage path, it is observed that a longer length of plug is provided than is necessary for structural strength purposes. Note that knowledge of pore-water pressure behaviour within a concrete plug is very limited. Therefore, a pressure gradient is assumed to exist from the hydrostatic pressure at the face in contact with the impounded water to zero at the other end. Garrett and Campbell (1958) proposed an approach, which is based on the observation that the resistance of the plug to passage of water along its contact with rock or through the adjacent fractured rock depends on the length of the plug and the resistance of the rock to the passage of water. They observed that these two factors are interrelated by the pressure gradient through the rock as the linking medium. They also determined the minimum length of the plug if the contact between plug and rock was un-grouted, as follows:

\[ L = \text{Pressure gradient among two faces} \]
\[ = \frac{20.8 \text{lb/ inch}^2}{\text{ft} (470 \text{kPa/m})} \]

**Selected Plug Arrangement**

It was determined that a 20 ft (6.1 m) long plug was necessary to satisfy the above criteria.

However, due to the uncertainties associated with the shaft and rock conditions and the importance of long-term performance of the plugs, an additional factor of safety of 1.5 was applied to the plug lengths. Provisions were also made for grouting of the surrounding rocks and rock/concrete interface as additional safety measures.

**Concrete mix designs**

Considerations are made to provide the right balance of ingredients to suit the particular mode of transportation and placing being used and to make sure that the optimum design is achieved. In addition to strength, the most important factor for the mix design is to obtain the correct workability. For this underground work, it is essential that high workability mixes be used. A mix design is, therefore, developed by the inclusion of plasticizing admixtures for transporting and placing. Also, cement replacement materials to minimize the thermal effects, which are discussed below:

**Thermal and shrinkage effects**

It is preferable to pour a concrete plug in one operation to avoid construction joints, which may form potential leakage paths through the plug. However, the large volume of concrete required for mass filling [310 cu.yd (237 m³) for shaft #1 and 422 cu. Yd. (322 m³) for shaft #2] can be subjected to detrimental thermal effects during setting. The resulting shrinkage is dependent upon the amount of cement included in the mix. Internal build up of heat due to the cement hydration process induces a high thermal stress. On cooling, thermal cracking may result and the integrity of the structure would be impaired. The three elements for effective temperature control program are used for the project, namely, cement content control, where particular type and amount of cement material (Type IV cement with fly ash and 2 inch (5.1cm) course aggregate) is used to lessen the heat generating potential of the concrete. Secondly, pre-cooling, where cooling of ingredients (set of 50°F by removing approximately 25,000 btu from the concrete components using liquid nitrogen) achieves a lower concrete temperature as placed in the plug, and finally post-cooling, where removing heat from the concrete with embedded cooling coils limit the temperature rise in the plug. Fig. 3 shows the post cooling measures for the plug concrete to limit the concrete temperature rise as minimal as possible. It was also decided to cast the concrete in two lifts in order to reduce the stresses in the supporting formwork and to provide more effective cooling systems. Chilled water is to be circulated for 7 days after lift 1 is placed. Plug cooling is to be continued for 21 days or until the ambient temperature of the surrounding rock is reached.
Control of water during plug construction
A crucial element of the design of the shaft plugs was control of the seepage into the mine and shafts during the construction of the plugs. The design incorporated a drainage collection system and temporary pumps to keep the construction area dry until the plugs could be concreted and grouted. Fig. 4 shows the water collection system during the construction of the shaft 2 plugs. Back up system was specified to ensure safety of workers in the shafts. Design provisions were also made for the dewatering of the shafts for the upper plugs, should the shaft flood and fill before construction of the upper plugs is completed.

Fig. 3. Post Cooling Measure for the Plug Concrete at No. 2 shaft
Grouting and mine stabilization

The following describes the design for the stabilization of the Detroit Salt Mine, which includes design of the stabilization measures in the mine, to ensure long-term mine stability, to prevent long-term leakage into the mine and subsequent solution of the salt. The design includes grouting in and around the shaft plugs including two-stage cement and chemical curtain grouting in the rock surrounding the plug and high-pressure contact grouting at the concrete-soil and concrete-rock interface. The intent of the grouting program was to stabilize the mine floor levels A and B, reduce the permeability of the rock surrounding the plug and to fill all voids created as a result of concrete placement or shrinkage.

A comprehensive grouting program to be executed include, cement and chemical grouting.
Cement grouting
This include consolidation and backfill grouting of voids and unstable zones below pillars and concrete piers level A and level B in the mine, primary curtain grouting at the concrete plugs in both shaft Nos. 1 and 2, and sealing and grouting of holes, penetrations, dewatering pipes, coolant pipes and grouting tubes at and in the shaft plugs. Fig. 5 shows the extent of new voids developed in mine flow level A since 1980 repair and grout pipe installed in void to inject cement grout to fill the voids. It also includes exploratory holes to be used as grout holes if required. Fig. 6 shows the voids at base and below piers in mine floor level B. Also shown are the grout pipes to fill the voids with grout and grout hole to extend 25 ft. (7.63 m) below base of pier. Also shown some detail of grout hole locations, and approximate area of salt pillars requiring restoration by means of grout placement. Grouting also includes voids at base and below piers. Fig. 7 shows some typical sections at B salt level with approximate area of salt pillars requiring restoration by means of grout placements. Grout holes to extend approximately 45 ft. below the mine flow. Fig. 8 and 9 show primary curtain grouting at the concrete plugs in shafts 1 and 2 lower plugs respectively. These holes are 2 inch (5.1 cm) in diameter and 20 ft. (6.1m) deep inside the rock making approximately 10 degrees with the horizontal.

Fig. 5. Plan showing the extent of voids and mine stabilization measure at "A" floor level
Fig. 6. Plan showing the extent of voids and mine stabilization measure at “B” floor level
Fig. 7a. Typical sections at "B" salt level with approximate area of salt pillars requiring restoration
Fig. 7b. Typical sections at "B" salt level with approximate area of salt pillars requiring restoration
Fig. 8. Primary curtain grouting at the concrete plugs in shafts 1 lower plug
Fig. 9. Primary curtain grouting at the concrete plugs in shafts 2 lower plug
GROUT HOLES TO FILL THE VOIDS AS SHOWN ON THE DRAWINGS AND AS MODIFIED BY THE INVESTIGATION PROGRAMS WERE CARRIED OUT. STAGE GROUTING TECHNIQUES WERE PLANNED. MAXIMUM LENGTH OF GROUT STAGE WILL BE 10 FT (3.05 M) UNLESS OTHERWISE MUTUALLY AGREED BETWEEN CONTRACTOR AND ENGINEER. THERE ARE VARIOUS APPROACHES TO EVALUATE THE SUITABILITY AND EFFECTIVENESS OF CEMENT-BASED SUSPENSION GROUTS INJECTED IN A PARTICULAR FORMATION IN SOIL OR ROCK GROUTING. THE AMENABILITY THEORY, NOW COMMONLY USED IN NORTH AMERICA (RAVEN 1992), WILL BE USED AS THE BASIS OF ASSESSING THE SUITABILITY AND EFFECTIVENESS OF THE SUSPENSION GROUTING SPECIFIED HEREIN. GROUT WILL BE MIXED IN BATCHES OF SUITABLE SIZE TO PERMIT GROUT FORMULATIONS TO BE VARIED QUICKLY TO SUIT THE RESPONSE OF THE FORMATION AND ACHIEVE MAXIMUM AMENABILITY.

GROUTING MIXES WILL BE USED FOR VOID FILING AT MINE LEVEL. SINCE SOME APPLICATIONS OF THIS GROUT ARE INTENDED TO BE RESTRICTED OR CONFINED TO A PREDETERMINED LATERAL SPREAD, DEPENDING ON THE AMENABILITY OF THE FORMATION AND THE AMOUNT OF GROUT ALREADY INJECTED INTO THAT SPECIFIC ZONE, A VISCOSITY MODIFIER WOULD BE USED, ONLY MARGINALLY INFLUENCING THE GROUT RHEOLOGY AT HIGH-SHEAR RATES, BUT CAUSING A SUBSTANTIAL INCREASE IN VISCOSITY IN THE CEMENT GROUT WHEN MOVING SLOWLY OR AT REST. FURTHERMORE, A RETARDER, SUCH AS POLYCAST LSR, OR APPROVED EQUIVALENT, WILL BE USED TO ALLOW THE FORMATION TO BE RECESS AFTER 24 HOURS (RAVEN 1992).

ALL GROUT HOLES / ZONES WILL BE GROUTED TO REFUSAL. A ZONE OR HOLE IS BROUGHT TO REFUSAL ONCE THE GROUT FLOW RATE TO THE HOLE / ZONE HAS BEEN REGISTERED TO BE LESS THAN 0.910 GALLONS PER MINUTE (GPM) FOR A MINIMUM OF 20 MINUTES AT THE MAXIMUM ALLOWABLE EFFECTIVE GROUTING PRESSURE. MAXIMUM ALLOWABLE GROUTING PRESSURE FOR A HOLE / ZONE WILL BE ONE PSI PER FOOT (22.6 KPA / M) OF DEPTH OR SURFACE COVER AT THE INTERVAL BEING GROUTED, OR AS MUTUALLY AGREED BETWEEN CONTRACTOR AND ENGINEER. IF A BACKPRESSURE EXISTS AFTER THE GROUTING OF A HOLE IS COMPLETED, CAP THE HOLE UNTIL THE PRESSURE FALLS TO A NEGligible AMOUNT.

CHEMICAL GROUTING
AFTER CONSTRUCTION OF THE PLUG AND COOLING TO AMBIENT TEMPERATURE, CONTACT GROUTING OF THE ROCK / CEMENT INTERFACE TAKES PLACE. IF WATER IS PRESENT, CONTACT GROUT WITH A WATER REACTIVE, FLEXIBLE, HYDROPHOBIC POLYURETHANE RESIN IS DONE. ON THE OTHER HAND, IF NO WATER IS PRESENT, CONTACT GROUT WITH A TWO COMPONENT POLYURETHANE ELASTOMER WITH HYDROPHILIC ANTENNAS IS CARRIED OUT. WHEN THE CONTACT GROUTING IS COMPLETE, A SECOND STAGE ROCK-GROUTING PROGRAM USING AN ACRYLAMIDE GROUT IN PREDRILLED HOLES SURROUNDING THE PLUG COMMENCES.

CONTACT GROUTING - CONTACT GROUTING OF SHAFT PLUGS INVOLVES INJECTION OF GROUT TO FILL ANNULAR SPACE AT ROCK / CONCRETE INTERFACE OR A CONCRETE CONSTRUCTION JOINT. A POROUS TUB ½ INCH (1.27CM) DIAMETER WITH SPIRAL STEEL REINFORCEMENT WILL BE USED AS CONTACT GROUT INJECTABLE TUBES TO PREVENT COLLAPSE DURING PLACEMENT OF CONCRETE AND WOVEN MEMBRANE, WHICH WILL NOT ALLOW PASSAGE OF CEMENT PARTICLES BUT WILL ALLOW EASY PASSAGE OF POLYURETHANE GROUT. STEEL TUBE ½ INCH (1.27 CM) INTERNAL DIAMETER RATED FOR 1,000 PSI (6890 KPA) INTERNAL PRESSURE COMPLETE WITH ALL CONNECTIONS, VALVES AND FITTINGS WILL BE USED FOR FEEDER TUBE FOR POLYURETHANE GROUT. ½ INCH (1.27 CM) DIAMETER STAINLESS STEEL PIPE RATED FOR TWO TIMES MAXIMUM PUMP PRESSURE COMPLETE WITH ALL CONNECTIONS, VALVES AND FITTINGS WILL BE USED FOR GROUT PIPE FOR ACRYLAMIDE GROUT. CONTACT GROUTING INVOLVES THE FOLLOWING PROCEDURE:

INJECTABLE GROUT TUBE WILL BE SECURED DIRECTLY TO ROCK OR CONCRETE SURFACE WITH FASTENING CLIPS AT MAXIMUM 6-INCH (15.2 CM) SPACING OR AS NECESSARY TO MAINTAIN TUBE IN CONTACT WITH ROCK OR CONCRETE DURING THE PLACING OF CONCRETE. SUFFICIENT TENSION TUBE WILL BE USED SO AS NOT TO ALLOW MOVEMENT OF THE TUBE DURING CONCRETE PLACEMENT WHILE MAINTAINING THE TUBE IN CONTACT WITH THE CONCRETE OR ROCK SURFACE. EACH INJECTABLE TUBE SHALL FORM A CONTINUOUS RING OR LOOP AROUND THE PLUG. OVERLAP IS MINIMUM OF 6 INCHES (15.2 CM).

AT LEAST ONE FEEDER TUBE WILL BE INSTALLED PER 12 FT (3.67M) OF INJECTABLE TUBE. THE TUBE SHALL BE PROPERLY CONNECTED TO INJECTABLE TUBE VIA T-PIECES, HEAT-SHRINK CONNECTIONS AS WELL AS MECHANICAL CONNECTIONS. EACH FEEDER TUBE WILL BE FASTENED TO A REINFORCING ROD TO PREVENT IT FROM BEING PULLED OUT OF ITS CONNECTION-FITTING WITH THE INJECTABLE TUBE. EACH FEEDER TUBE SHALL BE NUMBERED FOR EASY IDENTIFICATION.

GROUT WILL BE INJECTED INTO ONE END OF THE GROUT TUBE UNTIL UNDILUTED GROUT RETURN IS OBSERVED AT THE NEXT FEEDER TUBE. AT THIS TIME, THIS TUBE CAN BE BROUGHT ON LINE AND PRESSURIZED AS WELL, DURING A MULTIPLE HOLE-GROUTING PROGRAM. IF THE GROUT TRAVELS PROPERLY THROUGHOUT THE INJECTABLE TUBE FORMING ONE RING, SOME FEEDER TUBES OF THE FIRST GROUTED RING CAN BE DISCONNECTED AND THE VALVES SHUT. THESE PROCESS CONTINUES UNTIL ALL FEEDER TUBES, CONNECTION TO ONE RING, ARE ON LINE. GROUT UNTIL CONNECTION WITH THE ADJACENT RING HAS BEEN ESTABLISHED. AT THIS POINT, THE NEW RING IS BROUGHT ON LINE. GROUTING WILL BE CONTINUED UNTIL THE ENTIRE CONTACT ZONE HAS BEEN GROUTED AND GROUT IS EXPELLED ON BOTH ENDS OF THE PLUG.

SECONDARY CURTAIN GROUTING - SECONDARY CURTAIN GROUTING WILL BE PERFORMED AT SHAFT PLUGS, WHICH INVOLVES INJECTION OF CHEMICAL GROUT INTO ROCK MASS, TO FILL OPEN FRACTURES OR VOIDS IN THE ROCK STRUCTURE IN ORDER TO DECREASE THE PERMEABILITY OF THE ROCK MASS. THE INTENT OF THE SECONDARY CURTAIN GROUTING IS TO REDUCE THE ROCK MASS PERMEABILITY TO 1 X 10^-7 CM/sec.

THE FOLLOWING GROUT INGREDIENTS WILL BE USED: MONO - ACRYLAMIDE POWDER CONTAINING THE CROSS LINKER, ACTIVATOR - TRI-ETHANOL-AMINE (T+), INITIATOR - AMMONIUM PERSULFATE (AP), INHIBITOR - POTASSIUM FERRICYANIDE (KFeCN), BUFFER - SODIUM BICARBONATE, WATER AND DYE.
The following procedure will be followed:

Secondary curtain grouting will be performed with acrylamide grout at shaft bottom plugs after primary cementitious grouting is performed, concrete plug is placed, and contact grouting is completed. Grout pipes will be installed from secondary curtain grouting holes to 2 ft (0.61m) above plug concrete surface. Pipes at surface of plug will be leveled. Pipes will be secured to prevent movement during concreting and protect from damage. Grout pipes will be inserted 2 ft (0.61m) into the grout hole and the annulus between boreholes and grout pipe will be sealed with fast set cement mortar to prevent concrete from entering boreholes. Seal will be adequate to prevent the permeation of the contact-grout into the secondary grout holes. Surface of the mortar will be treated with polyurethane or epoxy coating to prevent permeation of the contact grout.

In-situ hydraulic conductivity test will be performed with real time monitoring. Gel time will be determined from the tests. Holes with similar in-situ permeability coefficient will be identified for multiple hole-grouting programs.

Summary and conclusions

Stabilization and design of the closure system of a salt mine in Detroit, U.S.A. was undertaken with a view to seal the shafts, so as to prevent leakage into the mine, preserve the mine stability and eliminate the long-term maintenance and repairs needed to provide safe access for inspection and pumping of water from the mine.

Concrete plugs were to be constructed below and above all potential leakage zones, after detailed geologic evaluation of the structural conditions of the surrounding rocks.

Provisions were to be made for cement and chemical grouting of the surrounding rocks and the rock/concrete interface, as additional safety measures. In-site hydraulic conductivity tests are to be performed with real time monitoring. Holes with similar in-situ permeability coefficient are to be identified for multiple grouting.

Detailed design is recently concluded and bid is open for the construction.

References


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